U.S. Department
of Transportation

**Federal Aviation
Administration**

Advisory Circular

Subject: AIRPORT PAVEMENT DESIGN AND EVALUATION
Date: 7/7/95
Initiated by: AAS-200
AC No: 150/5320-6D
Change:

1. PURPOSE. This advisory circular provides guidance to the public for the design and evaluation of pavements at civil airports.

2. CANCELLATION. AC 150/5320-6C, Airport Pavement Design and Evaluation, dated December 7, 1978, is canceled.

3. APPLICATION. The guidelines contained herein are recommended by the Federal Aviation Administration for applications at airports as appropriate.

4. RELATED READING MATERIAL. The publications listed in Appendix 4 provide further guidance and detailed information on the design and evaluation of airport pavements.

5. METRIC UNITS. To promote an orderly transition to metric units, this advisory circular includes both English and metric dimensions. The metric conversions may not be the exact equivalents, and until an official changeover to metric units is effected, the English dimensions will be used.

RAYMOND T. UHL
Acting Director, Office of Airport Safety and Standards

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FOREWORD

This advisory circular is intended to provide guidance on the structural design and evaluation of airport pavements.

Although aircraft landing gears are involved in airport pavement design and evaluation, this circular is not intended to dictate any facet of landing gear design. In 1958, the FAA adopted a policy of limiting maximum Federal participation in airport pavements to a pavement section designed to serve a 350,000-pound (159 000 kg) aircraft with a DC-8-50 series landing gear configuration. In addition, the intent of the policy was to insure that future aircraft were equipped with landing gears which would not stress Pavements more than the referenced 350,000-pound (159 000 kg) aircraft.

Aircraft manufacturers have accepted and followed the 1958 policy and have designed aircraft landing gear which conform to the policy even though aircraft gross weights have substantially exceeded 350,000 pounds (159 000 kg). This has been accomplished by increasing the number and spacing of landing gear wheels. This circular does not affect the 1958 policy with regard to landing gear design.

The pavement design guidance presented in Chapter 3 of this circular is based on methods of analysis which have resulted from experience and recent research. The change in methods was adopted to exploit these advances in pavement technology and thus provides better performing pavements and easier-to-use design curves. Generally speaking, the new design guidance will require somewhat thicker pavement sections than were required in the past.

The pavement evaluation portion of this circular is presented in Chapter 6 and is related back to the previous FAA method of design to insure continuity. An aircraft operator could be penalized unfairly if an existing facility were evaluated using a method different from that employed in the original design. A slight change in pavement thickness can have a dramatic effect on the payload or range of an aircraft. Since the new pavement design methodology generally requires slightly greater pavement thicknesses, an evaluation of an existing pavement using the new methodology would likely reduce allowable loads and penalize operators. To avoid this situation the evaluation should be based on the same methodology as was used for design.

CHAPTER 1.
AIRPORT PAVEMENTS - THEIR FUNCTION AND PURPOSES

100. GENERAL. Airport pavements are constructed to provide adequate support for the loads imposed by aircraft using an airport and to produce a firm, stable, smooth, all-year, all-weather surface free from dust or other particles that may be blown or picked up by propeller wash or jet blast. In order to satisfactorily fulfill these requirements, the pavement must be of such quality and thickness that it will not fail under the load imposed. In addition, it must possess sufficient inherent stability to withstand, without damage, the abrasive action of traffic, adverse weather conditions, and other deteriorating influences. To produce such pavements requires a coordination of many factors of design, construction, and inspection to assure the best possible combination of available materials and a high standard of workmanship.

a. **Types of Pavement**. Pavements discussed in this circular are flexible, rigid, hot mix asphalt overlays, and rigid overlays. Various combinations of pavement types and stabilized layers can result in complex pavements which would be classified in between flexible and rigid. The design and evaluation guidance in this circular can be adapted to any pavement type.

b. **Economic Analysis and Design Selection**. When properly designed and constructed, any pavement type (rigid, flexible, composite, etc.) can provide a satisfactory pavement for any civil aircraft. However, some designs may be more economical than others and still provide satisfactory performance. The engineer is required to provide a rationale for the selected design in the engineer's report (see AC 150/5300-9). Often this rationale will be based on economic factors derived from evaluating several design alternatives. Life-cycle cost analysis should be used if the design selection is based on least cost. An example of a life-cycle cost analysis of alternatives for pavement rehabilitation is shown in Appendix 1. More details on life-cycle cost analysis can be found in research report DOT/FAA/RD-81/78 (see Appendix 4). Many new developments in construction have evolved in recent times which can significantly affect pavement costs, such as, recycling. In instances where no clear cost advantage can be established in the design process, alternate bids should be taken. Design selection is not always controlled by economic factors. Operational constraints, funding limitations, future expansion, etc., can override economic factors in the design selection. These considerations should be addressed in the engineer's report.

c. **Pavement Courses.**

(1) **Surface**. Surface courses include portland cement concrete, hot mix asphalt, sand-bituminous mixture, and sprayed bituminous surface treatments.

(2) **Base**. Base courses consist of a variety of different materials which generally fall into two main classes, treated and untreated. The untreated bases consist of crushed or uncrushed aggregates. The treated bases normally consist of a crushed or uncrushed aggregate that has been mixed with a stabilizer such as cement, bitumen, etc.

(3) **Subbase**. Subbase courses consist of a granular material, a stabilized granular material, or a stabilized soil.

(4) **Geotextile**. Geotextiles are permeable, flexible, textile materials sometimes used to provide separation between pavement aggregate and the underlying subgrade. Geotextile needs and requirements within a pavement section are dependent upon subgrade soil and groundwater conditions and on the type of overlying pavement aggregate.

101. SPECIFICATIONS AND STANDARDS.

a. **Specifications**. Reference is made by Item Number throughout the text to construction material specifications contained in AC 150/5370-10, Standards for Specifying Construction of Airports.

b. **Geometric Standards**. Geometric standards concerning pavement lengths, widths, grades, and slopes are presented in advisory circulars listed in Appendix 4.

102. SPECIAL CONSIDERATIONS. Airport pavements should provide a surface which is not slippery and will provide good traction during any weather conditions. AC 150/5320-12, Measurement, Construction and Maintenance of Skid Resistant Airport Pavement Surfaces, presents information on skid resistant surfaces.

103. STAGE CONSTRUCTION OF AIRPORT PAVEMENTS. In some instances it may be necessary to construct the airport pavement in stages; that is, to build up the pavement profile, layer by layer, as the traffic using the facility increases in weight and number. Lateral staging, i.e., planning for future widening of pavements is sometimes advantageous to accommodate larger aircraft. If stage construction is to be undertaken, the need for sound planning cannot be overemphasized. The complete pavement should be designed prior to the start of any stage, and each stage must provide an operational surface. The planning of a stage constructed pavement should recognize a number of considerations.

a. Economics. Careful economic studies are required to determine if staged construction is warranted. Construction materials and labor costs follow inflationary trends and can be expected to increased as later stages are constructed. The costs and time involved in any pavement shutdown or diversion of traffic necessitated by the construction of any stage should be considered. The costs of mobilizing construction equipment several times should be compared with mobilizing once. The costs of maintaining an intermediate stage should be considered.

b. Adequacy of Each Stage. Each stage should be designed to adequately accommodate the traffic which will use the pavement until the next stage is constructed.

c. Drainage. The underlying layers and drainage facilities of a stage constructed pavement should be built to the standards required for the final cross section. Providing the proper foundation and drainage facilities in the first stage is mandatory as the underlying layers will not be readily accessible for upgrading in the future.

d. Communication. All parties concerned and, insofar as practicable, the general public should be informed that staged construction is planned. Staged construction sometimes draws unjust criticism when relatively new facilities are upgraded for the next stage.

CHAPTER 2. SOIL INVESTIGATIONS AND EVALUATION

200. GENERAL. The importance of accurate identification and evaluation of pavement foundations cannot be overemphasized. Although it is impossible to explore the entire field of soil mechanics in a publication such as this, the following text will highlight those aspects which are particularly important to the airport paving engineer.

a. Classification System. The Unified Soil Classification (USC) system should be used in engineering matters concerning civil airport pavements. To avoid misunderstanding, certain terms employed are defined below:

(1) Definition. For engineering purposes, and particularly as it applies to airports, soil includes all natural deposits which can be moved with earth moving equipment, without requiring blasting under unfrozen conditions. Harder materials are considered to be rock.

(2) Conditions and Properties. Soil conditions include such items as the elevation of the water table, the presence of water bearing strata, and the field properties of the soil. Field properties of the soil include the soil's density, moisture content, and frost penetration.

(3) Profile. The soil profile is the vertical arrangement of layers of soils, each of which possesses different physical properties from the adjacent layer.

(4) Subgrade. Subgrade soil is that soil which forms the foundation for the pavement. It is the soil directly beneath the pavement structure.

b. Costs. Soil conditions and the local prices of suitable construction materials are important items affecting the cost of construction of airport pavements. Earthwork and grading costs are directly related to the difficulty with which excavation can be accomplished and compaction obtained.

c. Subgrade Support. It should be remembered that the subgrade soil ultimately provides support for the pavement and the imposed loads. The pavement serves to distribute the imposed load to the subgrade over an area greater than that of the tire contact area. The greater the thickness of pavement, the greater is the area over which the load on the subgrade is distributed. It follows, therefore, that the more unstable the subgrade soil, the greater is the required area of load distribution and consequently the greater is the required thickness of pavement. The soils having the best engineering characteristics encountered in the grading and excavating operations should be incorporated in the upper layers of the subgrade by selective grading if economically feasible.

d. Drainage. In addition to the relationship which soil conditions bear to grading and paving operations, they determine the necessity for underdrains and materially influence the amount of surface runoff. Thus, they have a resulting effect on the size and extent of other drainage structures and facilities. (See FAA publication, AC 150/5320-5, Airport Drainage.)

201. SOIL INVESTIGATIONS.

a. Distribution and Properties. To provide essential information on the various types of soils, investigations should be made to determine their distribution and physical properties. This information combined with data on site topography and area climatic records, provides basic planning material essential to the logical and effective development of the airport. An investigation of soil conditions at an airport site will include:

(1) Survey. A soil survey to determine the arrangement of different layers of the soil profile with relation to the proposed subgrade elevation.

(2) Sampling. Collection of representative samples of the layers of soil.

(3) Testing. Testing of samples to determine the physical properties of the various soil materials with respect to in-place density and subgrade support.

(4) **Availability.** A survey to determine the availability of materials for use in construction of the subgrade and pavement.

b. **Procedures.** With respect to sampling and surveying procedures and techniques, ASTM D 420, Investigating and Sampling Soils and Rock for Engineering Purposes, is one of the most frequently used. This method is based entirely on the soil profile. In the field, ASTM D 2488, Description of Soils (Visual-Manual Procedures) is commonly used to identify soils by such characteristics as color, texture, structure, consistency, compactness, cementation, and to varying degrees, chemical composition.

(1) **Maps.** The use of Department of Agriculture soils maps, United States Geodetic Survey (USGS) geologic maps, and USGS engineering geology maps can prove valuable aids in the study of soils at and in the vicinity of the airport. Although the pedological classification, determined from these maps, does not treat soil as an engineering or construction material, data so obtained are extremely useful to the agronomist in connection with the development of turf areas on airports and to the engineer concerned with preliminary investigations of site selection, development costs, and alignment.

(2) **Aerial Photography.** The practice of determining data on soils by use of aerial photographs is established and commonly acceptable. Relief, drainage, and soil patterns may be determined from the photographs, and an experienced photo interpreter can define differences in characteristics of soils. By employing this method of investigation, it is possible to expedite soil studies and reduce the amount of effort required to gather data.

202. SURVEYING AND SAMPLING.

a. **Soil Borings.** The initial step in an investigation of soil conditions is a soil survey to determine the quantity and extent of the different types of soil, the arrangement of soil layers, and the depth of any subsurface water. These profile borings are usually obtained with a soil auger or similar device. Washed borings are not recommended due to inaccuracies of depth determinations. The intent of the borings is to determine the soil or rock profile and its lateral extent. Inasmuch as each location presents its particular problems and variations, the spacing of borings cannot always be definitely specified by rule or preconceived plan. Suggested criteria for the location, depth, and number of borings are given in Table 2-1. Wide variations in these criteria can be expected due to local conditions.

TABLE 2-1. RECOMMENDED SOIL BORING SPACINGS AND DEPTHS

AREA	SPACING	DEPTH
Runways and Taxiways	Random across pavement at 200 foot (68 m) intervals	Cut Areas - 10' (3.5 m) Below Finished Grade Fill Areas - 10' (3.5 m) Below Existing Ground[1]
Other Areas of Pavement	1 Boring per 10,000 Square Feet (930 sq m) of Area	Cut Areas - 10' (3.5 m) Below Finished Grade Fill Areas - 10' (3.5 m) Below Existing Ground[1]
Borrow Areas	Sufficient Tests to Clearly Define the Borrow Material	To Depth of Borrow Excavation

[1]For deep fills, boring depths shall be sufficient to determine the extent of consolidation and/or slippage the fill may cause.

b. **Number of Borings, Locations, and Depths.** Obviously, the locations, depths, and numbers of borings must be such that all important soil variations can be determined and mapped. Whenever past experience at the location in question has indicated that settlement or stability in deep fill areas may be a problem or, if in the opinion of the engineer, additional investigations are warranted, more or deeper borings may be required in order that the proper design, location, and construction procedures may be determined. Conversely, where uniform soil conditions are encountered, fewer borings may be acceptable.

c. Boring Log. A graphic log of soil conditions can be of great value in assessing subgrade conditions. It is recommended that the graphic log be developed which summarizes the results of the soil explorations. A typical graphic log is included as Figure 2-1. The graphic log should include:

(1) **Location**

(2) **Date Performed**

(3) **Type of exploration**

(4) **Surface elevation**

(5) **Depth of materials**

(6) **Sample identification numbers**

(7) **Classification**

(8) **Water table**

d. Soil Survey Areas. The soil survey is not confined to soils encountered in grading or necessarily to the area within the boundaries of the airport site. Possible sources of locally available material that may be used as borrow areas or aggregate sources should be investigated.

e. Undisturbed Samples. Samples representative of the different layers of the various soils encountered and various construction material discovered should be obtained and tested in the laboratory to determine their physical and engineering properties. In-situ properties such as in-place density, shear strength, consolidation characteristics, etc. may necessitate obtaining "undisturbed" core samples. ASTM D 1587, Thin Walled Tube Sampling of Soils, describes a method of obtaining "undisturbed" soil samples. Because the results of a test can only be as good as the sampling, it is of utmost importance that each sample be representative of a particular type of soil material and not be a careless and indiscriminate mixture of several materials.

f. Inplace Testing. Pits, open cuts, or both may be required for making inplace bearing tests, for the taking of undisturbed samples, for charting variable soil strata, etc. This type of supplemental soil investigation is recommended for situations which warrant a high degree of accuracy or when in situ conditions are complex and require extensive investigation.

203. SOIL TESTS.

a. Physical Soil Properties. To determine the physical properties of a soil and to provide an estimate of its behavior under various conditions, it is necessary to conduct certain soil tests. A number of field and laboratory tests have been developed and standardized. Detailed methods of performing soil tests are completely covered in publications of the American Society for Testing and Materials (ASTM).

b. Testing Requirements. Soil tests are usually identified by terms indicating the soil characteristics which the tests will reveal. Terms which identify the tests considered to be the minimum or basic requirement for airport pavement, with their ASTM designations and brief explanations, follow:

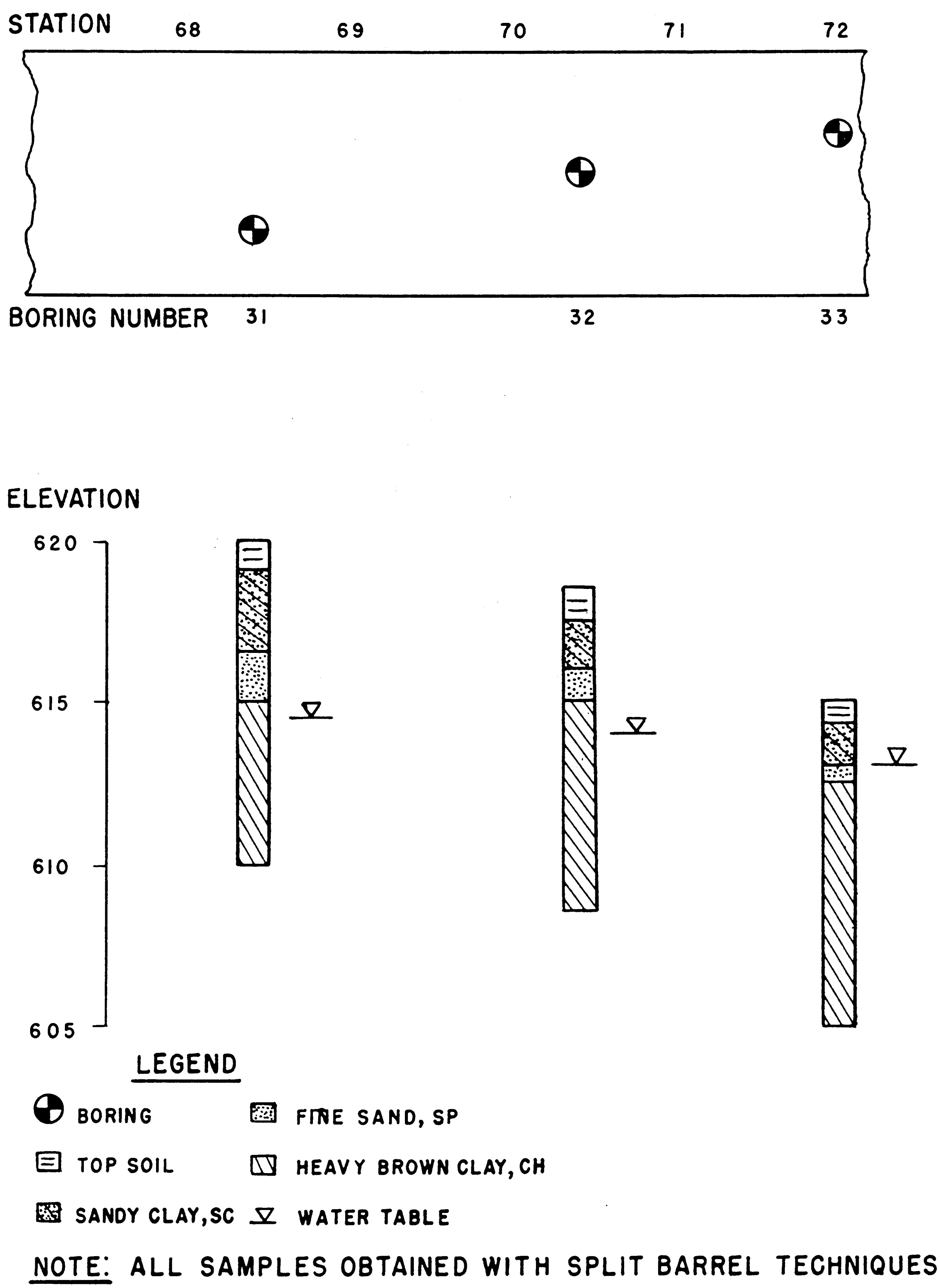


FIGURE 2-1 TYPICAL BORING LOG

(1) **Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants (ASTM D 421)** or **Wet Preparation of Soil Samples for Grain-Size Analysis and Determination of Soil Constants (ASTM D 2217).** The dry method (D-421) should be used only for clean, cohesionless granular materials. The wet method (D-2217) should be used for all cohesive or borderline materials. In case of doubt, the wet method should be used.

(2) **Particle-Size Analysis of Soils (ASTM C 422).** This analysis provides a quantitative determination of the distribution of particle sizes in soils.

(3) **Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D 4318).** The plastic and liquid limits of soil define in a standard manner the lowest moisture contents at which a soil will change from a semi-solid to a plastic state and at which a solid passes from a plastic to a liquid state, respectively. The plasticity index is the numerical difference between the plastic limit and the liquid limit. It indicates the range in moisture content over which a soil remains in a plastic state prior to changing into a liquid. The plastic limit, liquid limit and plasticity index of soils are used in engineering classification in accordance with the Unified Soil Classification System (ASTM D 2487). In conjunction with particle size analysis, natural moisture content and other soil properties or conditions, the limits may be used to estimate engineering properties or behavior of soils such as shrink/swell potential, consolidation characteristics, construction/stabilization characteristics, permeability, and strength characteristics.

(4) **Moisture-Density Relations of Soils (ASTM D 698, D 1557).** For purposes of compaction control during construction, tests to determine the moisture-density relations of the different types of soils should be performed.

(i) **Heavy Load Pavements.** For pavements designed to serve aircraft weighing 30,000 pounds (13 000 kg) or more, use ASTM Method D 1557.

(ii) **Light Load Pavements.** For pavements designed to serve aircraft weighing less than 30,000 pounds (13 000 kg), use ASTM Method D 698.

(5) **Bearing Ratio of Laboratory-Compacted Soils (ASTM D 1883).** This test is used to assign a California Bearing Ratio (CBR) value to subgrade soils for use in the design of flexible pavements.

(6) **Modulus of Soil Reaction (AASHTO T 222).** This test is used to determine the modulus of soil reaction, K, for use in the design of rigid pavements.

c. **Supplemental Tests.** In many cases additional soil tests will be required over those listed in paragraph 203b above. It is not possible to cover all the additional tests which may be required; however, a few examples are presented below. This list should not be considered all inclusive.

(1) **Shrinkage Factors of Soils (ASTM D 427).** This test may be required in areas where swelling soils might be encountered.

(2) **Permeability of Granular Soils (ASTM D 2434).** This test may be needed to assist in the design of subsurface drainage.

(3) **Determination of Organic Material in Soils by Wet Combustion (AASHTO T-194).** This test may be needed in areas where deep pockets of organic material are encountered or suspected.

(4) **California Bearing Ratio, Field In-place Tests (Mil-Std 621, Method 101).** Field bearing tests can be performed when the in site conditions satisfy density and moisture conditions which will exist under the pavement being designed. The method is also described in Manual Series No. 10, Soils Manual, The Asphalt Institute, College Park, MD.

204. UNIFIED SOIL CLASSIFICATION SYSTEM.

a. **Purpose.** The standard method of classifying soils for engineering purposes is ASTM D 2487, commonly called the Unified system. The primary purpose in determining the soil classification is to enable the engineer to predict probable field behavior of soils. The soil constants in themselves also provide some guidance on which to base performance predictions. The Unified system classifies soils first on the basis of grain size, then further subgroups soils on the plasticity constants. Table 2-2 presents the classification of soils by the Unified system.

b. **Initial Division.** As indicated in Table 2-2, the initial division of soils is based on the separation of coarse-and fine-grained soils and highly organic soils. The distinction between coarse and fine grained is determined by the amount of material retained on the No. 200 sieve. Coarse-grained soils are further subdivided into gravels and sands on the basis of the amount of material retained on the No. 4 sieve. Gravels and sands are then classed according to whether or not fine material is present. Fine-grained soils are subdivided into two groups on the basis of liquid limit. A separate division of highly organic soils is established for materials which are not generally suitable for construction purposes.

TABLE 2-2. CLASSIFICATION OF SOILS FOR AIRPORT PAVEMENT APPLICATIONS

MAJOR DIVISIONS		GROUP SYMBOLS	
Coarse-grained Soils more than 50% retained on No. 200 sieve[1]	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW GP
		Gravels with Fines	GM GC
	Sands less than 50% of coarse fraction retained on No. 4 sieve	Clean Sands	SW SP
		Sands with Fines	SM SC
Fine-grained Soils 50% or less retained on No. 200 sieve[1]	Silts and Clays Liquid Limit 50% or less		ML CL OL
	Silts and Clays Liquid Limit Greater than 50%		MH CH OH
Highly Organic Soils			PT

[1]Based on the material passing the 3-in (75-mm) sieve.

c. **Soil Groups.** Soils are further subdivided into 15 different groupings. The group symbols and a brief description of each is given below:

(1) GW Well-graded gravels and gravel-sand mixtures, little or no fines.
(2) GP Poorly graded gravels and gravel-sand mixtures, little or no fines.
(3) GM Silty gravels, gravel-sand-silt mixtures.
(4) GC Clayey gravels, gravel-sand-clay mixtures.
(5) SW Well-graded sands and gravelly sands, little or no fines.
(6) SP Poorly graded sands and gravelly sands, little or no fines.
(7) SM Silty sands, sand-silt mixtures.
(8) SC Clayey sands, sand-clay mixtures.
(9) ML Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.
(10) CL Inorganic clays of low to medium plasticity, gravelly clays, silty clays, lean clays.
(11) OL Organic silts and organic silty clays of low plasticity.
(12) MH Inorganic silts, micaceous or diatomaceous fine sands or silts, plastic silts.

(13) **CH** Inorganic clays or high plasticity, fat clays.
(14) **OH** Organic clays of medium to high plasticity.
(15) **PT** Peat, muck and other highly organic soils.

d. Final Classification. Determination of the final classification group requires other criteria in addition to that given in Table 2-2. These additional criteria are presented in Figure 2-2 and have application to both coarse and fine grained soils.

e. Flow Chart. A flow chart which outlines the soil classification process has been developed and is included as Figure 2-3. This flow chart indicates the steps necessary to classify soils in accordance with ASTM D 2487.

f. Field Identification. ASTM D 2488, Description of Soils (Visual-Manual Procedure), presents a simple, rapid method of field identification of soils. This procedure provides techniques for classifying soils rather accurately with a minimum of time and equipment.

g. Characteristics as Pavement Foundations. A table of pertinent characteristics of soils used for pavement foundations is presented in Table 2-3. These characteristics are to be considered as approximate, and the values listed are generalizations which should not be used in lieu of testing.

205. EXAMPLES. The following examples illustrate the classification of soils by the Unified system. The classification process progresses through the flow chart shown in Figure 2-3.

a. Example 1. Assume a soil sample has the following properties and is to be classified in accordance with the Unified system.

(1) **Fines.** Percent passing No. 200 sieve = 98%.

(2) **Liquid Limit.** Liquid limit on minus 40 material 30%.

(3) **Plastic Limit.** Plastic limit on minus 40 material 10%.

(4) **Plasticity Chart.** Above "A" line, see Figure 2-2. The soil would be classified as CL, lean clay of low to medium plasticity. Table 2-3 indicates the material would be of fair to poor value as a foundation when not subject to frost action. The potential for frost action is medium to high.

b. Example 2. Assume a soil sample with the following properties is to be classified by the Unified system.

(1) **Fines.** Percent passing No. 200 sieve = 48%.

CLASSIFICATION CRITERIA		
Classification on basis of percentage of fines Less than 5% Pass No. 200 sieve GW, GP, SW, SP More than 12% Pass No. 200 sieve GM, GC, SM, SC 5% to 12% Pass No. 200 sieve Borderline Classification requiring use of dual symbols	$C_u = D_{60}/D_{10}$ Greater than 4 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Not meeting both criteria for GW	
	Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
	Atterberg limits plot above "A" line and plasticity index greater than 7	
	$C_u = D_{60}/D_{10}$ Greater than 6 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
	Not meeting both criteria for SW	
	Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
	Atterberg limits plot above "A" line and plasticity index greater than 7	

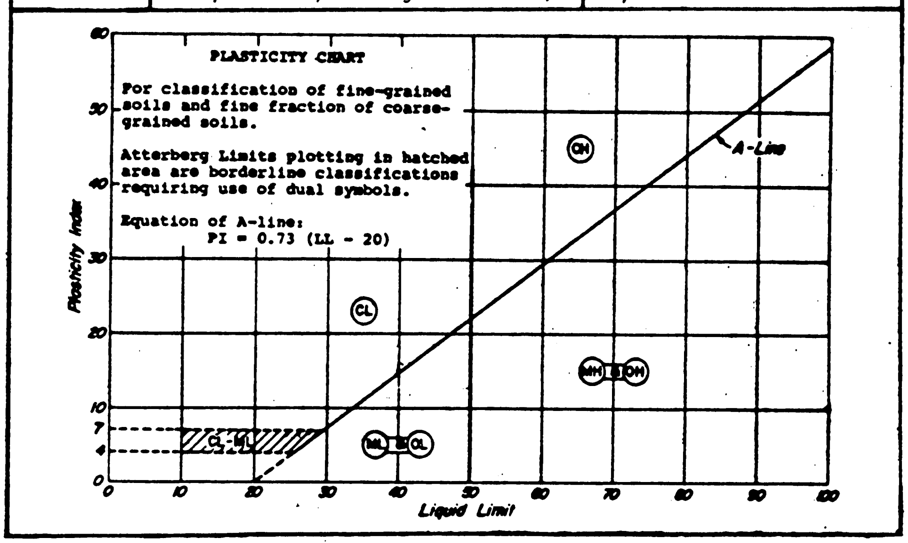
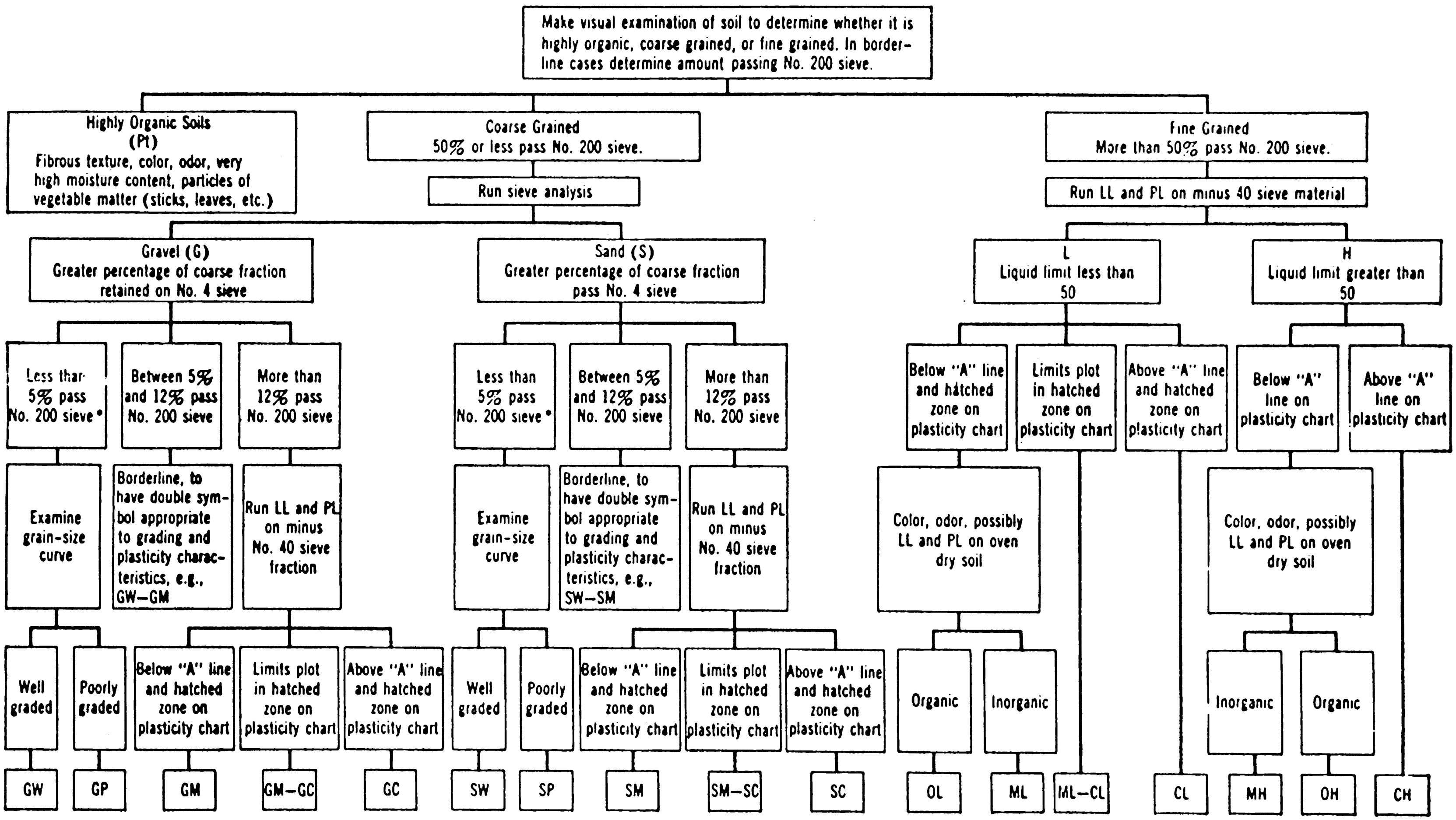


FIGURE 2-2 SOIL CLASSIFICATION CRITERIA



Note: Sieve sizes are U.S. Standard.

* If fines interfere with free-draining properties use double symbol such as GW-GM, etc.

Figure 2-3. Flow chart for Unified Soil Classification System

TABLE 2-3. Soil Characteristics Pertinent to Pavement Foundations

Major Divisions (1)	(2)	Letter (3)	Name (4)	Value as Foundation When Not Subject to Frost Action (5)	Value as Base Directly under Wearing Surface (6)	Potential Frost Action (7)	Compressibility and Expansion (8)	Drainage Characteristics (9)	Compaction Equipment (10)	Unit Dry Weight (pcf) (11)	Field CBR (12)	Subgrade Modulus k (pci) (13)
Coarse-grained soils	Gravel and gravelly soils	GW	Gravel or sandy gravel, well graded	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	125-140	60-80	300 or more
		GP	Gravel or sandy gravel, poorly graded	Good to excellent	Poor to fair	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	120-130	35-60	300 or more
		GU	Gravel or sandy gravel, uniformly graded	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	115-125	25-50	300 or more
		GM	Silty gravel or silty sandy gravel	Good to excellent	Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	130-145	40-80	300 or more
		GC	Clayey gravel or clayey sandy gravel	Good	Poor	Slight to medium	Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120-140	20-40	200-300
	Sand and sandy soils	SW	Sand or gravelly sand, well graded	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	110-130	20-40	200-300
		SP	Sand or gravelly sand, poorly graded	Fair to good	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	105-120	15-25	200-300
		SU	Sand or gravelly sand, uniformly graded	Fair to good	Not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	100-115	10-20	200-300
		SM	Silty sand or silty gravelly sand	Good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	120-135	20-40	200-300
		SC	Clayey sand or clayey gravelly sand	Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105-130	10-20	200-300
Fine-grained Soils	Low compressibility LL < 50	ML	Silts, sandy silts, gravelly silts, or diatomaceous soils	Fair to good	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	100-125	5-15	100-200
		CL	Lean clays, sandy clays, or gravelly clays	Fair to good	Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired equipment, sheepsfoot roller	100-125	5-15	100-200
		OL	Organic silts or lean organic clays	Poor	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired equipment, sheepsfoot roller	90-105	4-8	100-200
	High compressibility LL > 50	MH	Micaceous clays or diatomaceous soils	Poor	Not suitable	Medium to very high	High	Fair to poor	Rubber-tired equipment, sheepsfoot roller	80-100	4-8	100-200
		CH	Fat clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	90-110	3-5	50-100
		OH	Fat organic clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	80-105	3-5	50-100
Peat and other fibrous organic soils		Pt	Peat, humus and other	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical			

(2) **Gravel.** Percent of coarse fraction retained on No. 4 sieve = 70%.

(3) **Liquid Limit.** Liquid limit on minus 40 fraction = 60%.

(4) **Plastic Limit.** Plastic limit on minus 40 fraction = 20%.

(5) **Plasticity Index.** Compute Plasticity Index LL-PL = 40%.

(6) **Plasticity Chart.** Above "A" line, see Figure 2-2.

(7) **Classification.** This sample is classified as GC, clayey gravel. Table 2-3 indicates the material is good for use as a pavement foundation when not subject to frost action. The potential for frost action is slight to medium.

206. SOIL STRENGTH TESTS. Soil classification for engineering purposes provides an indication of the probable behavior of the soil as a pavement subgrade. This indication of behavior is, however, approximate. Performance different from that expected can occur due to a variety of reasons such as degree of compaction, degree of saturation, height of overburden, etc. The possibility of incorrectly predicting subgrade behavior can be largely eliminated by measuring soil strength. The strength of materials intended for use in flexible pavement structures is measured by the California Bearing Ratio (CBR) tests. Materials intended for use in rigid pavement structures are tested by the plate bearing method of test. Each of these tests is discussed in greater detail in the subsequent paragraphs.

a. California Bearing Ratio. The CBR test is basically a penetration test conducted at a uniform rate of strain. The force required to produce a given penetration in the material under test is compared to the force required to produce the same penetration in a standard crushed limestone. The result is expressed as a ratio of the two forces. Thus a material with a CBR value of 15 means the material in question offers 15% of the resistance to penetration that the standard crushed stone offers. Laboratory CBR tests should be performed in accordance with ASTM D 1883, Bearing Ratio of Laboratory-Compacted Soils. Field CBR tests should be conducted in accordance with the ASTM D 4429, Standard Test Method for Bearing Ratio of Soils in Place.

(1) **Laboratory.** Laboratory CBR tests are conducted on materials which have been obtained from the site and remolded to the density which will be obtained during construction. Specimens are soaked for 4 days to allow the material to reach saturation. A saturated CBR test is used to simulate the conditions likely to occur in a pavement which has been in service for some time. Pavement foundations tend to reach nearly complete saturation after about 3 years. Seasonal moisture changes also dictate the use of a saturated CBR design value since traffic must be supported during periods of high moisture such as spring seasons.

(2) **Field.** Field CBR tests can provide valuable information on foundations which have been in place for several years. The materials should have been in place for a sufficient time to allow for the moisture to reach an equilibrium condition. An example of this condition is a fill which has been constructed and surcharged for a long period of time prior to pavement construction.

(3) **Gravelly Materials.** CBR tests on gravelly materials are difficult to interpret. Laboratory CBR tests on gravel often yield CBR results which are too high due to the confining effects of the mold. The assignment of CBR values to gravelly subgrade materials may be based on judgment and experience. The information given in Table 2-3 may provide helpful guidance in selecting a design CBR value for a gravelly soil. Table 2-3 should not, however be used indiscriminately as a sole source of data. It is recommended that the maximum CBR for unstabilized gravel subgrade be 50.

(4) **Number of Tests.** The number of CBR tests needed to properly establish a design value cannot be simply stated. Variability of the soil conditions encountered at the site will have the greatest influence on the number of tests needed. As an approximate "rule of thumb" three CBR tests on each different major soil type should be considered. The preliminary soil survey will reveal how many different soil types will be encountered. The design CBR value should be conservatively selected. Common paving engineering practice is to select a value which is one standard

deviation below the mean. As a rule, a design CBR value of 3 the lowest practical value which should be assigned. In instances where the subgrade strength is lower than CBR = 3, the subgrade should be improved through stabilization or other means to raise the design CBR value.

(5) **Lime Rock Ratio**. Some areas of the country use the lime rock ratio, LBR, to express soil strength. To convert LBR to CBR, multiply LBR by 0.8.

b. **Plate Bearing Test**. As the name indicates, the plate bearing test measures the bearing capacity of the pavement foundation. The result, k value, can be envisioned as the pressure required to produce a unit deflection of the pavement foundation. The plate bearing test result, k value, has the units of pounds per cubic inch (Mega-Newtons per cubic meter). Plate bearing tests should be performed in accordance with the procedures contained in AASHTO T 222.

(1) **Sensitivity**. Rigid pavement design is not too sensitive to the k value. An error in establishing a k value will not have a drastic impact on the design thickness of the rigid pavement. Plate bearing tests must be conducted in the field and are best performed on test sections which are constructed to the design compaction and moisture conditions. A correction to the k value for saturation is required to simulate the moisture conditions likely to be encountered by the in-service pavement.

(2) **Number of Tests**. Plate bearing tests are relatively expensive to perform and thus the number of tests which can be conducted to establish a design value is limited. Generally only 2 or 3 tests can be performed for each pavement feature. The design k value should be conservatively selected.

(3) **Plate Size**. The rigid pavement design and evaluation curves presented in this circular are based on a k value determined by a static plate load test using a 30-inch (762 mm) diameter plate. Use of a plate of smaller diameter will result in a higher k value than is represented in the design and evaluation curves.

(4) **Subbase Effects**. It is recommended that plate bearing tests be conducted on the subgrade and the results adjusted to account for the effect of subbase. Figure 2-4 shows the increase in k value for various thicknesses of subbase over a given subgrade k. Plate bearing tests conducted on top of subbase courses can sometimes yield erroneous results since the depth of influence beneath a 30" inch (762 mm) bearing plate is not as great as the depth of influence beneath a slab loaded with an aircraft landing gear assembly. In this instance a subbase layer can influence the response of a bearing plate more than the response of a loaded pavement.

(5) **Stabilized Subbase**. The determination of k value for stabilized layers is a difficult problem. The k value normally has to be estimated. It is recommended that the k value be estimated as follows. The thickness of the stabilized layer should be multiplied by a factor ranging from 1.2 to 1.6 to determine the equivalent thickness of well-graded crushed aggregate. The actual value in the 1.2 - 1.6 range should be based on the quality of the stabilized layer and the thickness of the slab relative to the thickness of the stabilized layer. High quality materials which are stabilized with high percentages of stabilizers should be assigned an equivalency factor which is higher than a lower quality stabilized material. For a given rigid pavement thickness, a thicker stabilized layer will influence pavement performance more than a thin stabilized layer and should thus be assigned a higher equivalency factor.

(6) **Maximum k Value**. It is recommended that a design k value of 500 lbs/in^3 (136 MN/m^3) not be exceeded for any foundation. The information presented in Table 2-3 gives general guidance as to probable k values for various soil types.

c. **Additional Soil Strength Tests**. Where stability of the underlying section is questionable, additional soil strength tests may be necessary. Direct shear tests (ASTM D 3080) or field vane tests (ASTM D 2573) may be required to adequately design the pavement structure.

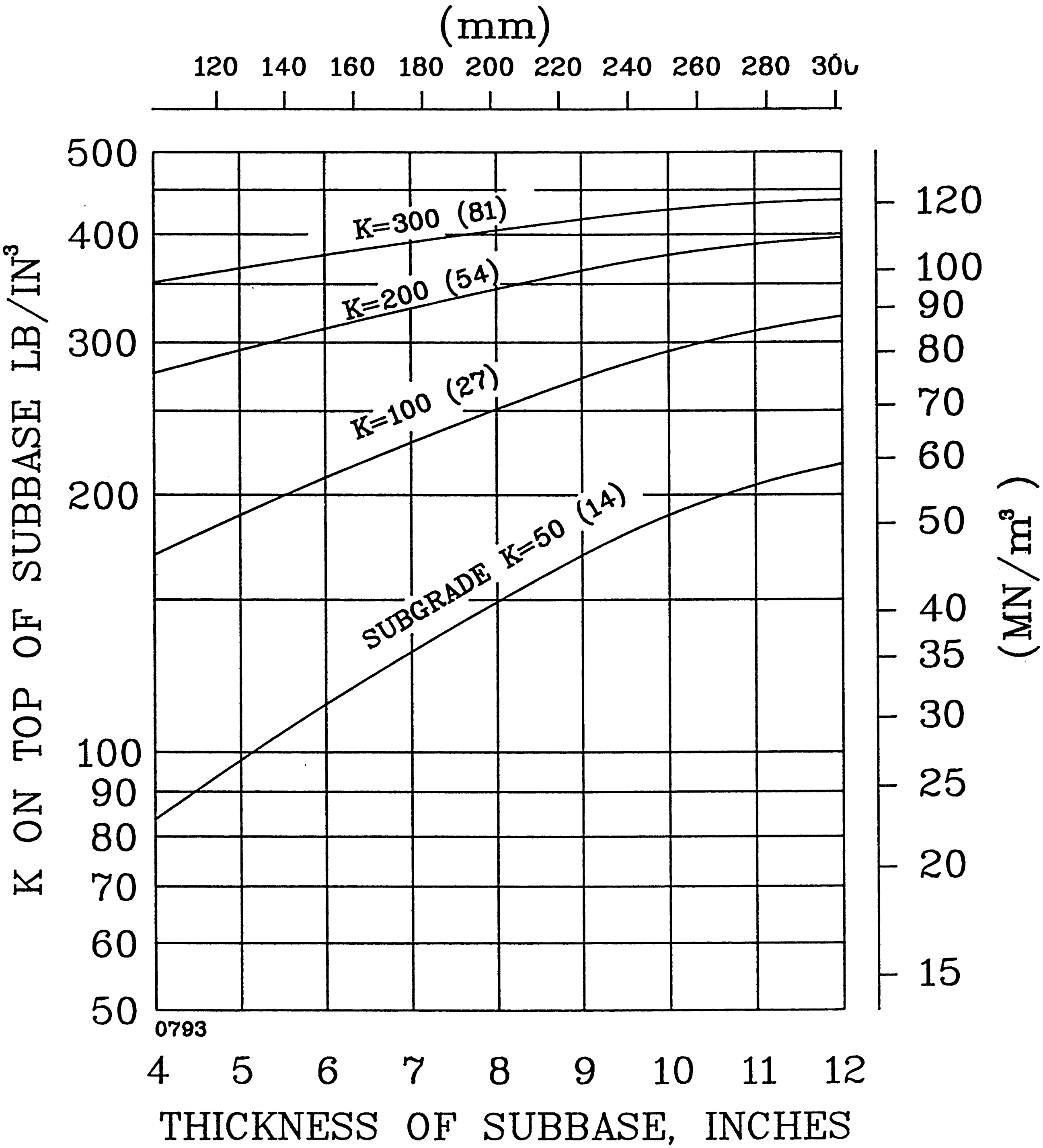


FIGURE 2-4. EFFECT OF SUBBASE ON MODULUS OF SUBGRADE REACTION

207. SUBGRADE STABILIZATION. Subgrade stabilization should be considered if one or more of the following conditions exist: poor drainage, adverse surface drainage, frost, or need for a stable working platform. Subgrade stabilization can be accomplished through the addition of chemical agents or by mechanical methods.

a. Chemical Stabilization. Different soil types require different stabilizing agents for best results. The following publications are recommended to determine the appropriate type and amount of chemical stabilization for subgrade soils. US Army, Corps of Engineers, Technical Manual TM 5-818-2/AFM 88-6 Chapter 6 ; Technical Manual 5-825.2/AFM 88-6 Chapter 2; Technical Manual 5-824-3/AFM 88-6, Chapter 3; Soil Cement Handbook, Portland Cement Association; and The Asphalt Institute Manual Series MS-19, A Basic Asphalt Emulsion Manual.

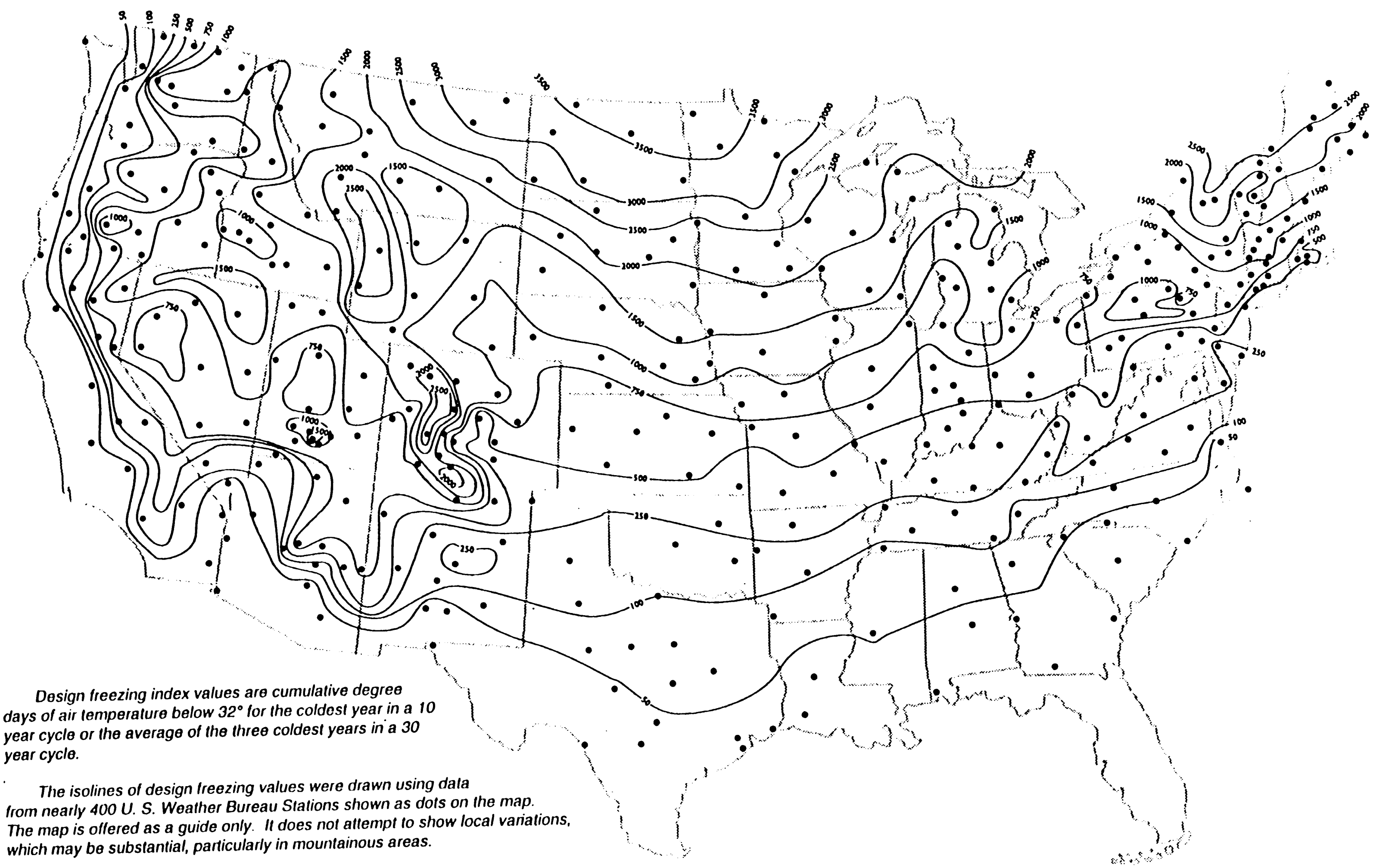
b. Mechanical Stabilization. In some instances subgrades cannot be adequately stabilized through the use of chemical additives. The underlying soils may be so soft that stabilized materials cannot be mixed and compacted over the underlying soils without failing the soft soils. Extremely soft soils may require bridging in order to construct the pavement section. Bridging can be accomplished with the use of thick layers of shot rock or cobbles. Thick layers of lean, porous concrete have also been used to bridge extremely soft soils. Geotextiles should be considered as mechanical stabilization over soft, fine-grained soils. Geotextiles can facilitate site access over soft soils and aid in reducing subgrade soil disturbance due to construction traffic. The geotextile will also function as a separation material to limit long-term weakening of pavement aggregate associated with contamination of the aggregate with underlying fine-grained soils. More information regarding construction over soft soils using geotextiles is provided in FHWA-KI-90-001 (see Appendix 4).

208. SEASONAL FROST. The design of pavements in areas subject to seasonal frost action requires special consideration. The detrimental effects of frost action may be manifested by nonuniform heave and in loss of soil strength during frost melting. Other related detrimental effects include: possible loss of compaction, development of pavement roughness, restriction of drainage, and cracking and deterioration of the pavement surface. Detrimental frost action requires three conditions be met simultaneously: first, the soil must be frost susceptible; secondly, freezing temperatures must penetrate into the frost susceptible soil; thirdly, free moisture must be available in sufficient quantities to form ice lenses.

a. Frost Susceptibility. The frost susceptibility of soils is dependent to a large extent on the size and distribution of voids in the soil mass. Voids must be of a certain critical size for the development of ice lenses. Empirical relationships have been developed correlating the degree of frost susceptibility with the soil classification and the amount of material finer than 0.02 mm by weight. Soils are categorized into four groups for frost design purposes, Frost Group 1 (FG-1), FG-2, FG-3, and FG-4. The higher the frost group number the more susceptible the soil, i.e., soils in frost group 4 are more frost susceptible than soils in frost groups 1, 2, or 3. Table 2-4 defines the frost groups.

b. Depth of Frost Penetration. The depth of frost penetration is a function of the thermal properties of the pavement and soil mass, the surface temperature, and the temperature of the pavement and soil mass at the start of the freezing season. Several methods are available to calculate the depth of frost penetration and subsurface temperatures. The method presented here is a simplification of a method based on the modified Berggren equation. This method requires the use of the air freezing index and the dry unit weight of the soil.

(1) Air Freezing Index. The air freezing index is a measure of the combined duration and magnitude of below freezing temperatures occurring during any given freezing season. the average daily temperature is used in the calculation of freezing index. For example, assume the average daily temperature is 10 degrees below freezing for 10 days. The freezing index would be calculated as follows, 10 degrees X 10 days = 100 degree days. Ideally, air freezing indices should be based on actual data obtained from a meteorological station located in close proximity to the construction site. The air freezing index used for design (design air freezing index) should be based on the average of the 3 coldest winters in a 30 year period, if available, or the coldest winter observed in a 10 year period. Figures 2-6 and 2-7 show the approximate design air freezing indices for the lower United States and Alaska, respectively. The values shown in Figures 2-6 and 2-7 do not show local variation which may be substantial, especially in mountainous areas.



Design freezing index values are cumulative degree days of air temperature below 32° for the coldest year in a 10 year cycle or the average of the three coldest years in a 30 year cycle.

The isolines of design freezing values were drawn using data from nearly 400 U. S. Weather Bureau Stations shown as dots on the map. The map is offered as a guide only. It does not attempt to show local variations, which may be substantial, particularly in mountainous areas.

The actual design freezing index used should be computed for the specific project using temperature data from station nearest site. See text.

FIGURE 2-5. DISTRIBUTION OF DESIGN AIR FREEZING INDEXES IN THE CONTINENTAL U.S.

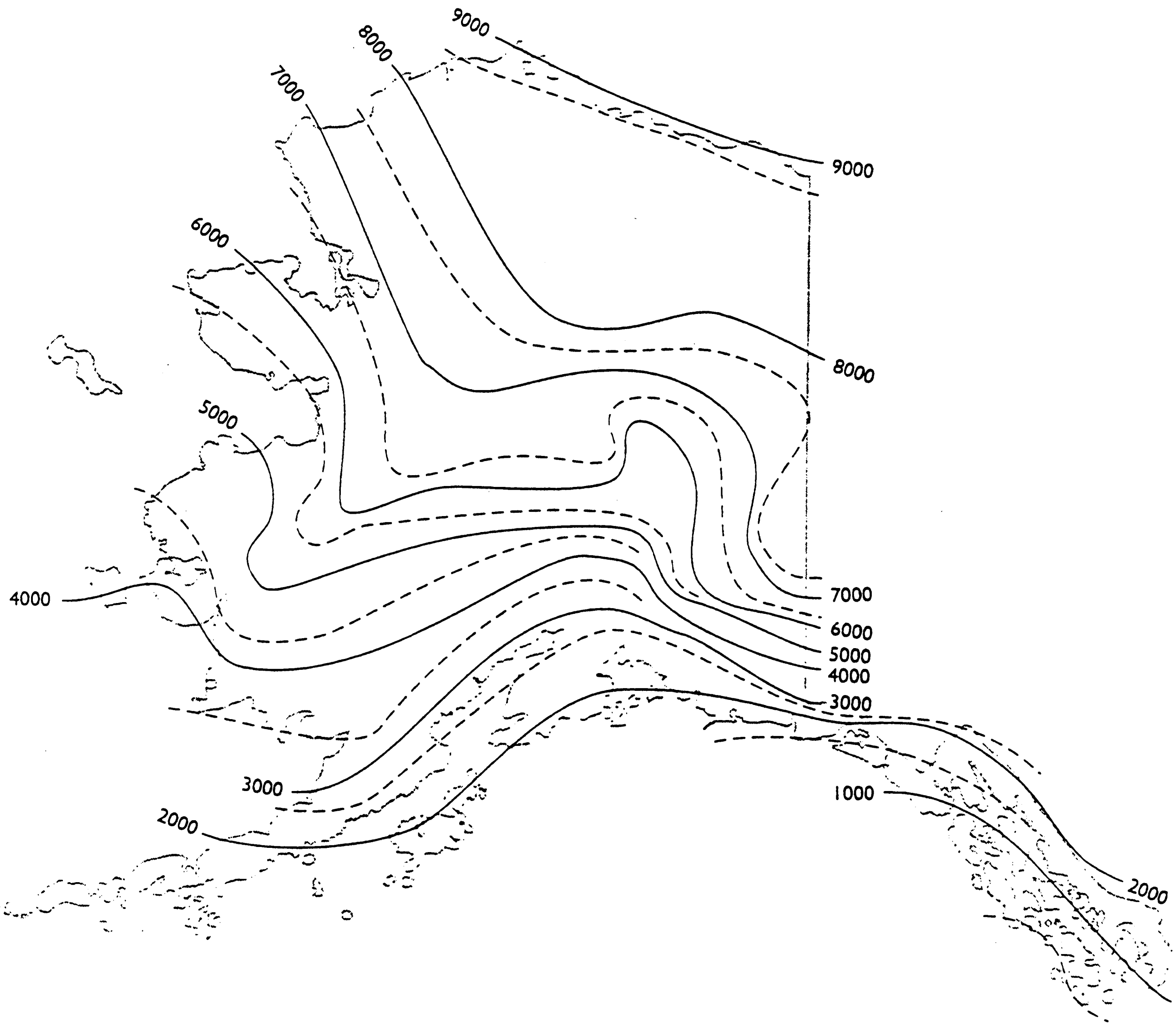


FIGURE 2-6. DISTRIBUTION OF DESIGN AIR FREEZING INDEX VALUES IN ALASKA

(2) **Depth of Frost Penetration.** The relationship between air freezing index and depth of frost penetration is shown in Figure 2-8. The thermal properties of the soil mass are reflected by differences in the dry unit weight of the subgrade soil. In the development of this method, the pavement is assumed to be either a 12 inch (300 mm) thick rigid pavement or a 20 inch (510 mm) thick flexible pavement. The depths of frost penetration shown on Figure 2-8 are measured from the pavement surface downward.

TABLE 2-4. SOIL FROST GROUPS

Frost Group	Kind of Soil	Percentage finer than 0.02 mm by weight	Soil Classification
FG-1	Gravelly Soils	3 to 10	GW, GP, GW-GM, GP-GM
FG-2	Gravelly Soils	10 to 20	GM, GW-GM, GP-GM,
	Sands	3 to 15	SW, SP, SM, SW-SM SP-SM
FG-3	Gravelly Soils	Over 20	GM, GC
	Sands, except very fine silty sands	Over 15	SM, SC
	Clays, PI above 12	-	CL, CH
FG-4	Very fine silty sands	Over 15	SM
	All Silts	-	ML, MH
	Clays, PI = 12 or less	-	CL, CL-ML
	Varied Clays and other fine grained banded sediments.	-	CL, CH, ML, SM

c. **Free Water.** The availability of free water in the soil mass to freeze and form ice lenses is the third consideration which must be present for detrimental frost action to occur. Water may be drawn from considerable depths by capillary action, by infiltration from the surface or sides, or by condensation of atmospheric water vapor. Generally speaking, if the degree of saturation of the soil is 70% or greater, frost heave will probably occur. For all practical purposes, the designer should assume that sufficient water to cause detrimental frost action will be present.

d. **Frost Design.** The design of pavements to offset seasonal frost effects is presented in Chapter 3. A more detailed and rigorous discussion of frost action and its effects can be found in Research Report No. FAA/RD/74/30, see Appendix 3.

209. PERMAFROST. In arctic regions soils are often frozen at considerable depths year round. Seasonal thawing and refreezing of the upper layer of permafrost can lead to severe loss of bearing capacity and/or differential heave. In areas with continuous high-ice-content permafrost at shallow depths, satisfactory pavements are best ensured by restricting seasonal thawing to the pavement and to a non-frost susceptible base course. This approach is intended to prevent degradation (thawing) of the permafrost layer.

a. **Susceptibility.** The frost susceptibility of soils in permafrost areas is classified the same as given above in paragraph 206.

b. **Depth of Thaw Penetration.** Pavement design for permafrost areas must consider the depth of seasonal thaw penetration. The depth to which thawing temperatures penetrate into permafrost may be estimated using Figure 2-9. Use of Figure 2-9 requires inputs air thawing index, average wind speed during the thaw period, pavement type, and density of the permafrost layer. The air thawing index is expressed in degree days and is the difference between average daily temperature and 32 degrees Fahrenheit (0 degrees Celsius) multiplied by the number of days the temperature exceeds freezing. The thawing index used for design (design thawing index) should be based on the 3 warmest summers in the last 30 years of record. If 30 year records are not available, data from the warmest summer in the latest 10 year period may be used.

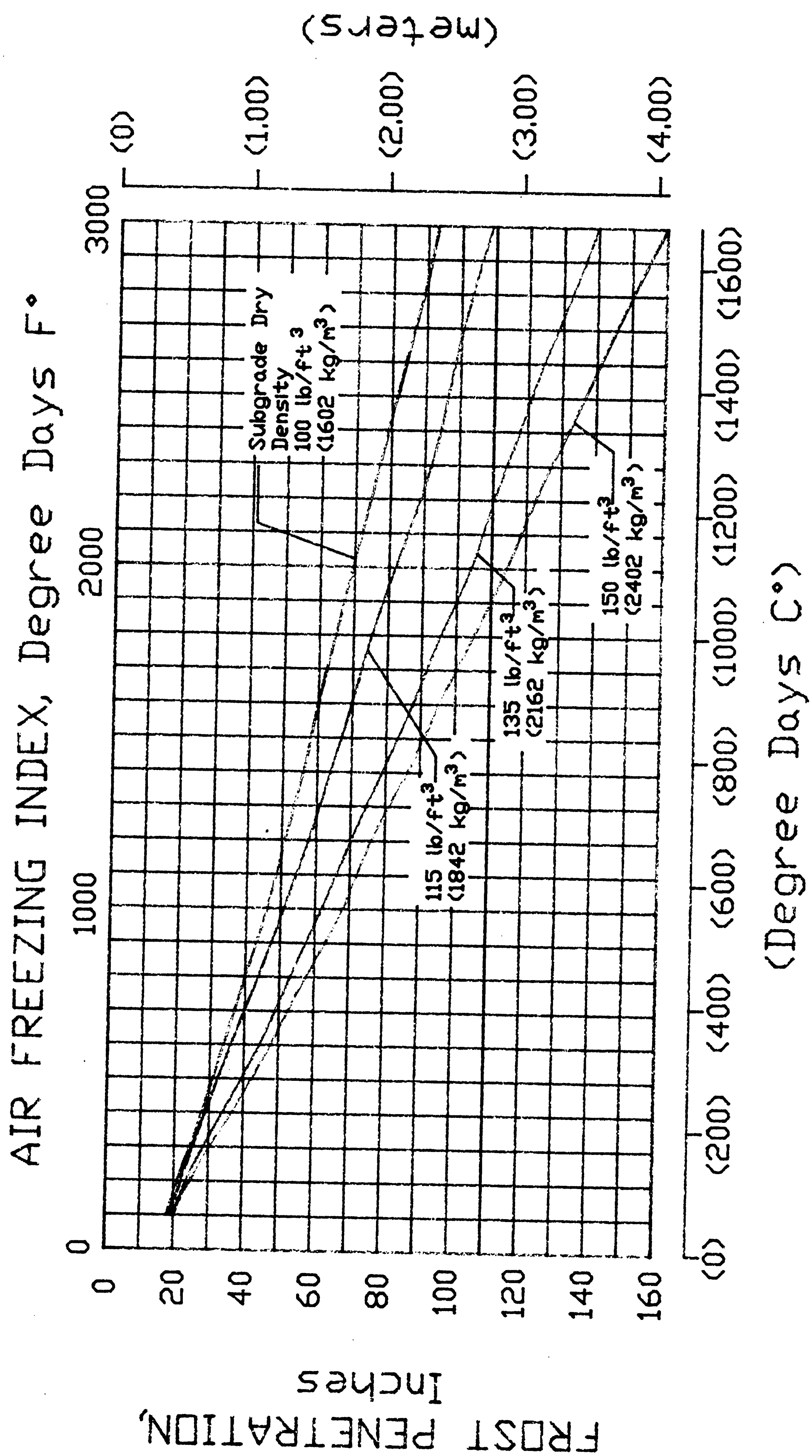
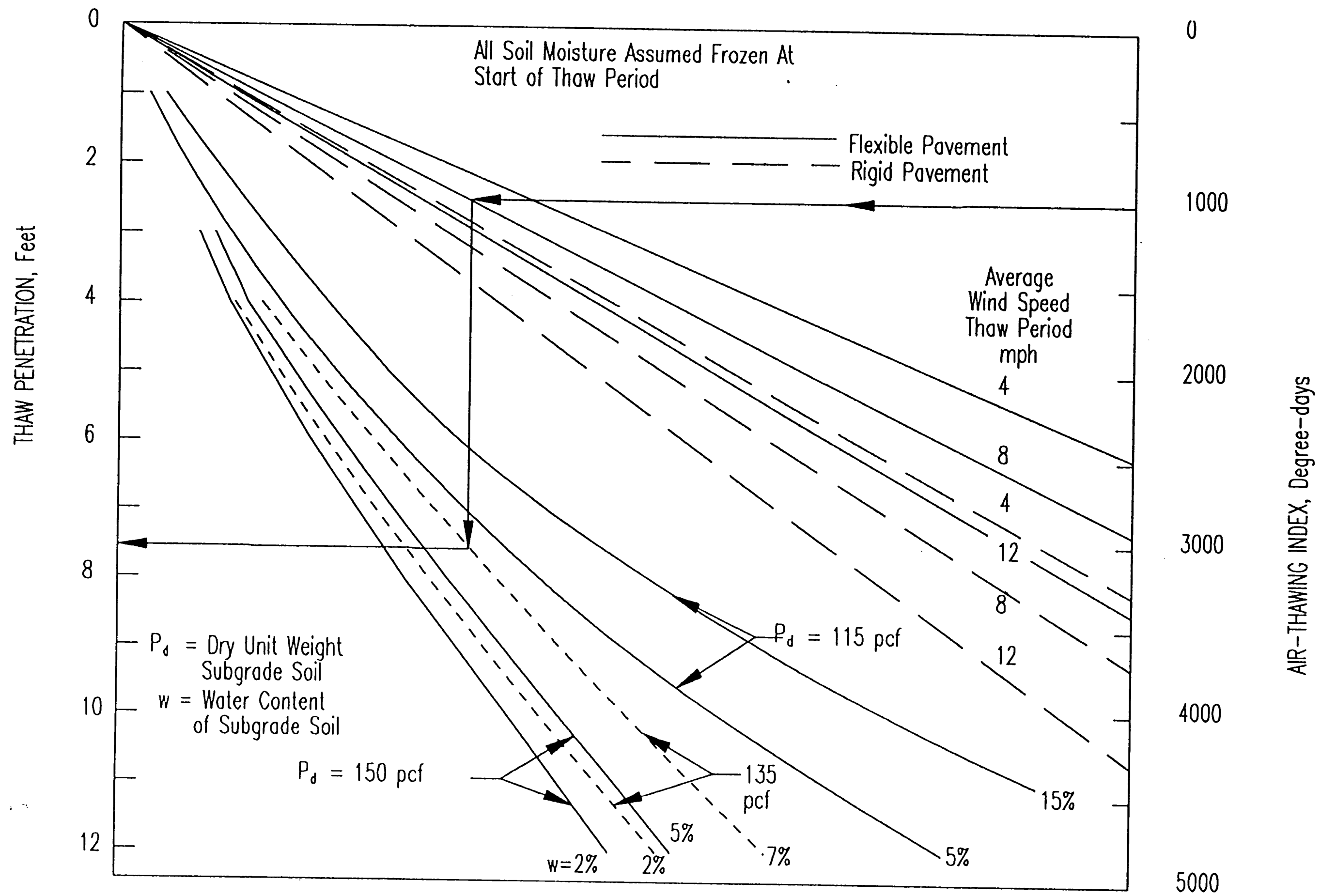


FIGURE 2-7. DEPTH OF FROST PENETRATION



RELATIONSHIP BETWEEN AIR THAWING INDEX AND THAW

Note:
1 foot = 0.3048 m
1 pcf − 16.02 kg/m³
1 mph = 1.609 km/h

FIGURE 2-8. RELATIONSHIP BETWEEN AIR THAWING INDEX AND THAW PENETRATION INTO GRANULAR, NON-FROST SUSCEPTABLE SUBGRADE SOIL

c. Muskeg. Muskeg is sometimes encountered in arctic areas. Muskeg is a highly organic soil deposit which is essentially a swamp. Every effort should be made to avoid pavement construction on this material. If construction in areas of muskeg is unavoidable and the soil survey shows the thickness of muskeg is less than 5 feet (1.5 m), the muskeg should be removed and replaced with granular fill. If the thickness of muskeg is too great to warrant removal and replacement, a 5 foot (1.5 m) granular fill should be placed over the muskeg. These thicknesses are based on experience and it should be anticipated that differential settlement will occur and considerable maintenance will be required to maintain a smooth surface. Use of a geotextile between the muskeg surface and the bottom of granular fill is recommended to prevent migration of the muskeg up into the granular fill. In this application, the geotextile is considered to perform the function of separation. Additional information on the design and construction of geotextiles performing the separation function within pavement sections is provided in FHWA-HI-90-001 (see Appendix 4).

d. Permafrost Design. Design of pavements in areas of permafrost is discussed in Chapter 3. Further information on permafrost can be found in Research Report No. FAA/RD/74/30, see Appendix 4.

CHAPTER 3. PAVEMENT DESIGN

SECTION 1. DESIGN CONSIDERATIONS

300. SCOPE. This chapter covers pavement design for airports serving aircraft with gross weights of 30,000 pounds (13 000 kg) or more. Chapter 5 is devoted to the design of pavements serving lighter aircraft with gross weights under 30,000 pounds (13 000 kg).

301. DESIGN PHILOSOPHY. The FAA policy of treating the design of aircraft landing gear and the design and evaluation of airport pavements as three separate entities is described in the Foreword to this advisory circular. The design of airport pavements is a complex engineering problem which involves a large number of interacting variables. The design curves presented in this chapter are based on the CBR method of design for flexible pavements and a jointed edge stress analysis for rigid pavements. Other design procedures such as those based on layered elastic analysis and those developed by The Asphalt Institute and the Portland Cement Association may be utilized to determine pavement thicknesses when approved by the FAA. These procedures will yield slightly different design thicknesses due to different basic assumptions. All pavement designs should be summarized on FAA Form 5100-1, Airport Pavement Design, which is considered to be part of the Engineer's Report. An Engineer's Report should be prepared for FAA review and approval along with initial plans and specifications. Because of thickness variations, the evaluation of existing pavements should be performed using the same method as was employed in the design. Procedures to be used in evaluating pavements are described in detail in Chapter 6 of this advisory circular. Details on the development of the FAA method of design are as follows:

a. Flexible Pavements. The flexible pavement design curves presented in this chapter are based on the California Bearing Ratio (CBR) method of design. The CBR design method is basically empirical; however, a great deal of research has been done with the method and reliable correlations have been developed. Gear configurations are related using theoretical concepts as well as empirically developed data. The design curves provide the required total thickness of flexible pavement (surface, base, and subbase) needed to support a given weight of aircraft over a particular subgrade. The curves also show the required surface thickness. Minimum base course thicknesses are given in a separate table. A more detailed discussion of CBR design is presented in Appendix 2.

b. Rigid Pavements. The rigid pavement design curves in this chapter are based on the Westergaard analysis of edge loaded slabs. The edge loading analysis has been modified to simulate a jointed edge condition. Pavement stresses are higher at the jointed edge than at the slab interior. Experience shows practically all load induced cracks develop at jointed edges and migrate toward the slab interior. Design curves are furnished for areas where traffic will predominantly follow parallel or perpendicular to joints and for areas where traffic is likely to cross joints at an acute angle. The thickness of pavement determined from the curves is for slab thickness only. Subbase thicknesses are determined separately. A more detailed discussion of the basis for rigid pavement design is presented in Appendix 2.

302. BACKGROUND. An airfield pavement and the operating aircraft represent an interactive system which must be addressed in the pavement design process. Design considerations associated with both the aircraft and the pavement must be recognized in order to produce a satisfactory design. Careful construction control and some degree of maintenance will be required to produce a pavement which will achieve the intended design life. Pavements are designed to provide a finite life and fatigue limits are anticipated. Poor construction and lack of preventative maintenance will usually shorten the service life of even the best designed pavement.

a. Variables. The determination of pavement thickness requirements is a complex engineering problem. Pavements are subject to a wide variety of loadings and climatic effects. The design process involves a large number of interacting variables which are often difficult to quantify. Although a great deal of research work has been completed and more is underway, it has been impossible to arrive at a direct mathematical solution of thickness requirements. For this reason the determination of pavement thickness must be based on the theoretical analysis of load distribution through pavements and soils, the analysis of experimental pavement data, and a study of the performance of pavements under actual service conditions. Pavement thickness curves presented in this chapter have been developed through correlation of the data obtained from these sources. Pavements designed in accordance with these standards are intended to provide a structural life of 20 years that is free of major maintenance if no major changes in forecast traffic are encountered. It is likely that rehabilitation of surface grades and renewal of skid resistant properties will be needed before 20 years due to destructive climatic effects and deteriorating effects of normal usage.

b. Structural Design. The structural design of airport pavements consists of determining both the overall pavement thickness and the thickness of the component parts of the pavement. There are a number of factors which influence the thickness of pavement required to provide satisfactory service. These include the magnitude and character of the aircraft loads to be supported, the volume of traffic, the concentration of traffic in certain areas, and the quality of the subgrade soil and materials comprising the pavement structure.

303. AIRCRAFT CONSIDERATIONS.

a. Load. The pavement design method is based on the gross weight of the aircraft. For design purposes the pavement should be designed for the maximum anticipated takeoff weight of the aircraft. The design procedure assumes 95 percent of the gross weight is carried by the main landing gears and 5 percent is carried by the nose gear. AC 150/5300-13, Airport Design, lists the weight of nearly all civil aircraft. Use of the maximum anticipated takeoff weight is recommended to provide some degree of conservatism in the design and is justified by the fact that changes in operational use can often occur and recognition of the fact that forecast traffic is approximate at best. By ignoring arriving traffic some of the conservatism is offset.

b. Landing Gear Type and Geometry. The gear type and configuration dictate how the aircraft weight is distributed to the pavement and determine pavement response to aircraft loadings. It would have been impractical to develop design curves for each type of aircraft. However, since the thickness of both rigid and flexible pavements is dependent upon the gear dimensions and the type of gear, separate design curves would be necessary unless some valid assumptions could be made to reduce the number of variables. Examination of gear configuration, tire contact areas, and tire pressure in common use indicated that these follow a definite trend related to aircraft gross weight. Reasonable assumptions could therefore be made and design curves constructed from the assumed data. These assumed data are as follows:

(1) **Single Gear Aircraft.** No special assumptions needed.

(2) **Dual Gear Aircraft.** A study of the spacing between dual wheels for these aircraft indicated that a dimension of 20 inches (0.51 m) between the centerline of the tires appeared reasonable for the lighter aircraft and a dimension of 34 inches (0.86 m) between the centerline of the tires appeared reasonable for the heavier aircraft.

(3) **Dual Tandem Gear Aircraft.** The study indicated a dual wheel spacing of 20 inches (0.51 m) and a tandem spacing of 45 inches (1.14 m) for lighter aircraft, and a dual wheel spacing of 30 inches (0.76 m) and a tandem spacing of 55 inches (1.40 m) for the heavier aircraft are appropriate design values.

(4) **Wide Body Aircraft.** Wide body aircraft; i.e., B-747, DC-10, and L-1011 represent a radical departure from the geometry assumed for dual tandem aircraft described in paragraph (c) above. Due to the large differences in gross weights and gear geometries, separate design curves have been prepared for the wide body aircraft.

c. Tire Pressure. Tire pressure varies between 75 and 200 PSI (516 to 1 380 kPa) depending on gear configuration and gross weight. It should be noted that tire pressure asserts less influence on pavement stresses as gross weight increases, and the assumed maximum of 200 PSI (1 380 kPa) may be safely exceeded if other parameters are not exceeded and a high stability surface course is used.

d. Traffic Volume. Forecasts of annual departures by aircraft type are needed for pavement design. Information on aircraft operations is available from Airport Master Plans, Terminal Area Forecasts, the National Plan of Integrated Airport Systems, Airport Activity Statistics and FAA Air Traffic Activity. These publications should be consulted in the development of forecasts of annual departures by aircraft type.

304. DETERMINATION OF DESIGN AIRCRAFT. The forecast of annual departures by aircraft type will result in a list of a number of different aircraft. The design aircraft should be selected on the basis of the one requiring the greatest pavement thickness. Each aircraft type in the forecast should be checked to determine the pavement thickness required by using the appropriate design curve with the forecast number of annual departures for that aircraft. The aircraft type which produces the greatest pavement thickness is the design aircraft. The design aircraft is not necessarily the heaviest aircraft in the forecast.

305. DETERMINATION OF EQUIVALENT ANNUAL DEPARTURES BY THE DESIGN AIRCRAFT.

a. Conversions. Since the traffic forecast is a mixture of a variety of aircraft having different landing gear types and different weights, the effects of all traffic must be accounted for in terms of the design aircraft. First, all aircraft must be converted to the same landing gear type as the design aircraft. Factors have been established to accomplish this conversion. These factors are constant and apply to both flexible and rigid pavements. They represent an approximation of the relative fatigue effects of different gear types. Much more precise and theoretically rigorous factors could be developed for different types and thicknesses of pavement. However, such precision would be impractical for hand calculation as numerous iterations and adjustments would be required as the design evolved. At this stage of the design process such precision is not warranted. The following conversion factors should be used to convert from one landing gear type to another:

To Convert From	To	Multiply Departures by
single wheel	dual wheel	0.8
single wheel	dual tandem	0.5
dual wheel	dual tandem	0.6
double dual tandem	dual tandem	1.0
dual tandem	single wheel	2.0
dual tandem	dual wheel	1.7
dual wheel	single wheel	1.3
double dual tandem	dual wheel	1.7

Secondly, after the aircraft have been grouped into the same landing gear configuration, the conversion to equivalent annual departures of the design aircraft should be determined by the following formula:

$$\log R_1 = \log R_2 \times \left(\frac{W_2}{W_1}\right)^{\frac{1}{2}}$$

where:
R_1 = equivalent annual departures by the design aircraft
R_2 = annual departures expressed in design aircraft landing gear
W_1 = wheel load of the design aircraft
W_2 = wheel load of the aircraft in question

For this computation 95 percent of the gross weight of the aircraft is assumed to be carried by the main landing gears. Wide body aircraft require special attention in this calculation. The procedure discussed above is a relative rating which compares different aircraft to a common design aircraft. Since wide body aircraft have significantly different landing gear assembly spacings than other aircraft, special considerations are needed to maintain the relative effects. This is done by treating each wide body as a 300,000-pound (136 100 kg) dual tandem aircraft when computing equivalent annual departures. This should be done in every instance even when the design aircraft is a wide body. After the equivalent annual departures are determined, the design should proceed using the appropriate design curve for the design aircraft. For example if a wide body is the design aircraft, all equivalent departures should be calculated as described above; then the design curve for the wide body should be used with the calculated equivalent annual departures.

b. Example: Assume an airport pavement is to be designed for the following forecast traffic:

Aircraft	Gear Type	Average Annual Departures	Maximum Takeoff Weight lbs.	(kg)
727-100	dual	3,760	160,000	(72 600)
727-200	dual	9,080	190,500	(86 500)
707-320B	dual tandem	3,050	327,000	(148 500)
DC-9-30	dual	5,800	108,000	(49 000)
CV-880	dual tandem	400	184,500	(83 948)
737-200	dual	2,650	115,500	(52 440)
L-1011-100	dual tandem	1,710	450,000	(204 120)
747-100	double dual tandem	85	700,000	(317 800)

(1) **Determine Design Aircraft.** A pavement thickness is determined for each aircraft in the forecast using the appropriate design curves. The pavement input data, CBR, K value, flexural strength, etc., should be the same for all aircraft. Aircraft weights and departure levels must correspond to the particular aircraft in the forecast. In this example the 727-200 requires the greatest pavement thickness and is thus the design aircraft.

(2) **Group Forecast Traffic into Landing Gear of Design Aircraft.** In this example the design aircraft is equipped with a dual wheel landing gear so all traffic must be grouped into the dual wheel configuration.

(3) **Convert Aircraft to Equivalent Annual Departures of the Design Aircraft.** After the aircraft mixture has been grouped into a common landing gear configuration, the equivalent annual departures of the design aircraft can be calculated.

Aircraft	Equi. Dual Gear Departs.	Wheel Load lbs.	Wheel Load (kg)	Wheel Load of Design Aircraft lbs.	Wheel Load of Design Aircraft (kg)	Equi. Annual Departs Design Aircraft
727-100	3,760	38,000	(17 240)	45,240	(20 520)	1,891
727-200	9,080	45,240	(20 520)	45,240	(20 520)	9,080
707-320B	5,185	38,830	(17 610)	45,240	(20 520)	2,764
DC-9-30	5,800	25,650	(11 630)	45,240	(20 520)	682
CV-880	680	21,910	(9 940)	45,240	(20 520)	94
737-200	2,650	27,430	(12,440)	45,240	(20 520)	463
747	145	35,625[1]	(16 160)	45,240	(20 520)	83
L-1011	2,907	35,625[1]	(16 160)	45,240	(20,520)	1,184
						Total = 16,241

[1]Wheel loads for wide body aircraft will be taken as the wheel load for a 300,000-pound (136 100 kg) aircraft for equivalent annual departure calculations.

(4) **Final Result.** For this example the pavement would be designed for 16,000 annual departures of a dual wheel aircraft weighing 190,500 pounds (86 500 kg). The design should, however, provide for the heaviest aircraft in the traffic mixture, B747-100, when considering depth of compaction, thickness of asphalt surface, drainage structures, etc.

c. **Other Methods**. More refined methods of considering mixed traffic are possible. These refined methods may consider variations in material properties due to climatic effects, take-off versus landing loads, aircraft tread dimensions, etc. Use of these refined methods is allowable under the conditions given in paragraph 301.

306. TRAFFIC DISTRIBUTION. Research studies have shown that aircraft traffic is distributed laterally across runways and taxiways according to statistically normal (bell shaped) distribution. FAA Report No. FAA-RD-36, Field Survey and Analysis of Aircraft Distribution on Airport Pavements, dated February 1975, contains the latest research information on traffic distribution. The design procedures presented in this circular incorporate the statistically normal distribution in the departure levels. In addition to the lateral distribution of traffic across pavements, traffic distribution and nature of loadings are considered for aprons, and high speed turnoffs.

307. **TYPICAL SECTIONS**. Airport pavements are generally constructed in uniform, full width sections. Runways may be constructed with a transversely variable section, if practical. A variable section permits a reduction in the quantity of materials required for the upper paving layers of the runway. However, more complex construction operations are associated with variable sections and are usually more costly. The additional construction costs may negate any savings realized from reduced material quantities. Typical plan and section drawings for transversely variable section runway pavements are shown in Figure 3-1. Deviations from these typical sections will be common due to the change inherent in staged construction projects where runways are extended and the location of taxiways is uncertain. As a general rule-of-thumb the designer should specify full pavement thickness T where departing traffic will be using the pavement; pavement thickness of 0.9T will be specified where traffic will be arrivals such as high speed turnoffs; and pavement thickness of 0.7T will be specified where pavement is required but traffic is unlikely such as along the extreme outer edges of the runway. Note that the full-strength keel section is 50 feet (15 m) on the basis of the research study discussed in paragraph 306a.

308. **FROST AND PERMAFROST DESIGN**. The design of an airport pavement must consider the climatic conditions which will act on the pavement during its construction and service life. The protection of pavements from the adverse effects of seasonal frost and permafrost effects are considered in the design of airport pavements as discussed below.

a. **Seasonal Frost**. The adverse effects of seasonal frost have been discussed in Chapter 2. The design of pavements in seasonal frost areas may be based on either of two approaches. The first approach is based on the control of pavement deformations resulting from frost action. Under this approach, sufficient combined thickness of pavement and non-frost-susceptible material must be provided to eliminate, or limit to an acceptable amount, frost penetration into the subgrade and its adverse effects. The second approach is based on providing adequate pavement load carrying capacity during the critical frost melting period. The second approach provides for the loss of load carrying capacity due to frost melting but ignores the effects of frost heave. Three design procedures have been developed which encompass the above approaches and are discussed below.

(1) **Complete Frost Protection**. Complete frost protection is accomplished by providing a sufficient thickness of pavement and non-frost-susceptible material to totally contain frost penetration. This method is intended to prevent underlying frost susceptible materials from freezing. To use the complete protection method , the depth of frost penetration is determined by the procedure given in Chapter 2. The thickness of pavement required for structural support is compared with the depth of frost penetration computed. The difference between the pavement thickness required for structural support and the computed depth of frost penetration is made up with non-frost-susceptible material. Depending on grades and other considerations, provision for complete protection may involve removal and replacement of a considerable amount of subgrade material. Complete frost protection is the most positive, and is usually the most costly, method of providing frost protection.

(2) **Limited Subgrade Frost Penetration**. The limited subgrade frost penetration method is based on holding frost heave to a tolerable level. Frost is allowed to penetrate a limited amount into the underlying frost susceptible subgrade. Sixty five (65%) of the depth of frost penetration is made up with non-frost-susceptible material. Use of the method is similar to the complete protection method. Additional frost protection is required if the thickness of the structural section is less than 65% of the frost penetration. The limited subgrade frost penetration method allows a tolerable (based on experience) amount of frost heave.

(3) **Reduced Subgrade Strength**. The reduced subgrade strength method is based on the concept of providing a pavement with adequate load carrying capacity during the frost melting period. This method does not consider the effects of frost heave. Use of the reduced subgrade strength method involves assigning a subgrade strength rating to the pavement for the frost melting period. The various soil frost groups as defined in Chapter 2, should be assigned strength ratings as shown below:

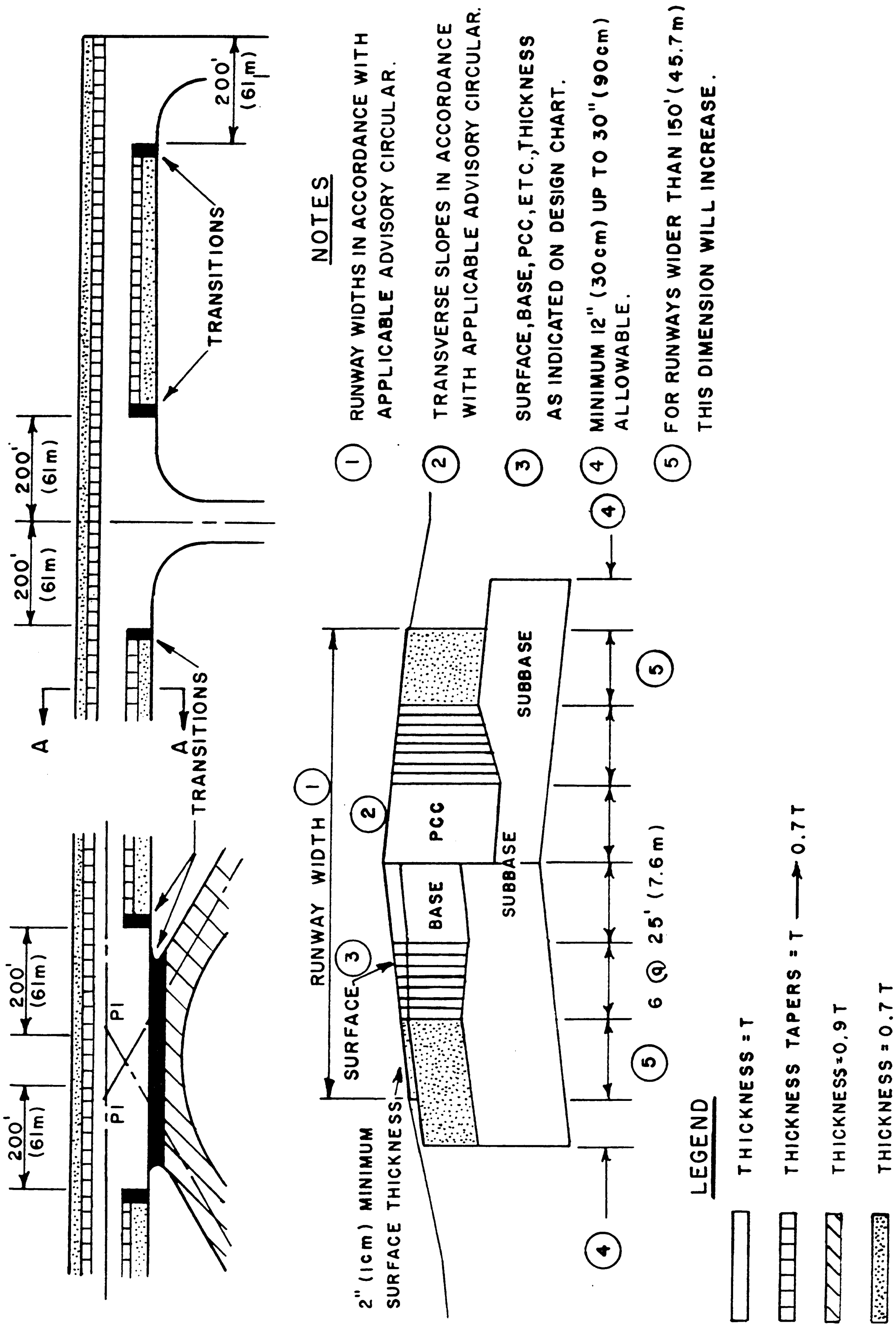


FIGURE 3-1. TYPICAL PLAN AND CROSS SECTION FOR RUNWAY PAVEMENT

TABLE 3-1. REDUCED SUBGRADE STRENGTH RATINGS

Frost Group	Flexible Pavement CBR Value	Rigid Pavement k-value
FG-1	9	50
FG-2	7	40
FG-3	4	25
FG-4	Reduced Subgrade Strength Method Does Not Apply	

The required pavement thicknesses are determined from the appropriate design curves using the reduced subgrade strength ratings. Pavement thicknesses thus established reflect the requirements for the subgrade in its weakened condition due to frost melting.

b. Applications. Due to economic considerations, the maximum practical depth of frost protection that should be provided is normally 72 inches (1.8 m). The recommended applications of the three methods of frost protection discussed above are as follows. In addition to these recommended applications, local experience should be given strong consideration when designing for frost conditions.

(1) **Complete Frost Protection.** The complete frost protection method applies only to FG-3 and FG-4 soils which are extremely variable in horizontal extent. These soil deposits are characterized by very large, frequent, and abrupt changes in frost heave potential. The variability is such that the use of transition sections is not practical.

(2) **Limited Subgrade Frost Penetration.** This design method should be used for FG-4 soils except where the conditions require complete protection, see (1) above. The method also applies to soils in frost groups FG-1, FG-2, and FG-3 when the functional requirements of the pavement permit a minor amount of frost heave. Consideration should be given to using transition sections where horizontal variability of frost heave potential permits.

(3) **Reduced Subgrade Strength.** The reduced subgrade strength method is recommended for FG-1, FG-2, and FG-3 subgrades which are uniform in horizontal extent or where the functional requirements of the pavement will permit some degree of frost heave. the method may also be used for variable FG-1 through FG-3 subgrades for less sensitive pavements which are subject to slow speed traffic and heave can be tolerated.

c. Permafrost. The design of pavements in permafrost regions must consider not only the effects of seasonal thawing and refreezing, but also the effects of construction on the existing thermal equilibrium. Changes in the subsurface thermal regime may cause degradation of the permafrost table, resulting in severe differential settlements drastic reduction of pavement load carrying capacity. Gravel surfaced pavements are rather common in permafrost areas and generally will provide satisfactory service. These pavements often exhibit considerable distortion but are rather easily regraded. The absence of a waterproof surface is not a great problem because these areas are usually have low precipitation. Three design methods for asphaltic or concrete surfaced pavements are discussed below.

(1) **Complete Protection Method.** The objective of the complete protection method is to ensure that the underlying permafrost remains frozen year round. Seasonal thawing is restricted to non-frost-susceptible materials. This method is analogous to the complete frost protection method of design for seasonal frost. The thickness of pavement required for structural support is first determined. The depth of seasonal thaw is then computed as described in Chapter 2 or using information based on local experience. The difference between the depth of seasonal thaw and the thickness needed for structural support is the amount of non-frost-susceptible material which must be provided to fully contain the depth of seasonal thaw. The use of relatively high moisture retaining soils, such as uniformly graded sands, should be considered. If some heaving can be tolerated, the use of frost-susceptible soils in the FG-1 or FG-2 groups may also be considered. If FG-1 or FG-2 soils are used, they must be placed so as to be as uniform as possible. Normally economic considerations will limit the depth of treatment to a maximum of 6 feet (1.8 m).

(2) **Reduced Subgrade Strength Method.** If conditions are such that the complete protection method of design is not practical, the design may be based on the reduced subgrade strength method. The use of this

method for permafrost design is identical to that presented in paragraph 308b(3) above. this method should provide a pavement with sufficient structural support during the seasonal permafrost thaw period but will likely result in differential heaving. If practical, it may be advisable to delay paving for 2 or 3 years to allow the embankment to reach equilibrium.

(3) **Insulating Panels.** A third approach which is not as common is the use of insulating panels beneath the pavement structure to protect against degradation of the permafrost. This method can lead to problems if the insulating panels are crushed by the weight of the overburden or by the live loads. Crushing of the cell structure of the insulation results in loss of insulating properties and failure to serve its intended purpose. Pavements using this technique must be very carefully constructed and may be subject to load limitations because of the need to guard against crushing the insulating panels. A significant change in the weight of using aircraft may fail the insulating panels. Since the FAA has no standards or design criteria for the use of insulating panels, their use on federally funded construction requires FAA approval on a case-by-case basis.

SECTION 2. FLEXIBLE PAVEMENT DESIGN

309. GENERAL. Flexible pavements consist of a hot mix asphalt wearing surface placed on a base course and, when required by subgrade conditions, a subbase. The entire flexible pavement structure is ultimately supported by the subgrade. Definitions of the function of the various components are given in the following paragraphs. For some aircraft the base and subbase should be constructed of stabilized materials. The requirements for stabilized base and subbase are also discussed in this section.

310. HOT MIX ASPHALT SURFACING. The hot mix asphalt surface or wearing course must prevent the penetration of surface water to the base course; provide a smooth, well-bonded surface free from loose particles which might endanger aircraft or persons; resist the shearing stresses induced by aircraft loads; and furnish a texture of nonskid qualities, yet not cause undue wear on tires. To successfully fulfill these requirements, the surface must be composed of mixtures of aggregates and bituminous binders which will produce a uniform surface of suitable texture possessing maximum stability and durability. Since control of the mixture is of paramount importance, these requirements can best be achieved by use of a central mixing plant where proper control can be most readily obtained. A dense-graded hot mix asphalt concrete such as Item P-401 produced in a central mixing plant will most satisfactorily meet all the above requirements. Whenever a hot mix asphalt surface is subject to spillage of fuel, hydraulic fluid, or other solvents; such as at aircraft fueling positions and maintenance areas, protection should be provided by a solvent resistant surface.

311. BASE COURSE. The base course is the principal structural component of the flexible pavement. It has the major function of distributing the imposed wheel loadings to the pavement foundation, the subbase and/or subgrade. The base course must be of such quality and thickness to prevent failure in the subgrade, withstand the stresses produced in the base itself, resist vertical pressures tending to produce consolidation and resulting in distortion of the surface course, and resist volume changes caused by fluctuations in its moisture content. In the development of pavement thickness requirements, a minimum CBR value of 80 is assumed for the base course. The quality of the base course depends upon composition, physical properties and compaction. Many materials and combinations thereof have proved satisfactory as base courses. They are composed of select, hard, and durable aggregates. Specifications covering the quality of components, gradation, manipulation control, and preparation of various types of base courses for use on airports for aircraft design loads of 30,000 pounds (14 000 kg) or more are as follows:

(1) Item P-208 - Aggregate Base Course[1]
(2) Item P-209 - Crushed Aggregate Base Course
(3) Item P-211 - Lime Rock Base Course
(4) Item P-304 - Cement Treated Base Course
(5) Item P-306 - Econocrete Subbase Course
(6) Item P-401 - Plant Mix Bituminous Pavements

[1]The use of Item P-208, Aggregate Base Course, as base course is limited to pavements designed for gross loads of 60,000 lbs. (27 000 kg) or less. When Item P-208 is used as base course the thickness of the hot mix asphalt surfacing should be increased 1 inch (25 mm) over that shown on the design curves.

312. SUBBASE. A subbase is included as an integral part of the flexible pavement structure in all pavements except those on subgrades with a CBR value of 20 or greater (usually GW or GP type soils). The function of the subbase is similar to that of the base course. However, since it is further removed from the surface and is subjected to lower loading intensities, the material requirements are not as strict as for the base course. In the development of pavement thickness requirements the CBR value of the subbase course is a variable.

a. **Quality**. Specifications covering the quality of components, gradations, manipulation control, and preparation of various types of subbase courses for use on airports for aircraft design loads of 30,000 pounds (14 000 kg) or more are as follows:

(1) Item P-154 - Subbase Course
(2) Item P-210 - Caliche Base Course
(3) Item P-212 - Shell Base Course

(4) Item P-213 - Sand Clay Base Course[1]
(5) Item P-301 - Soil Cement Base Course[1]

[1]Use of Items P-213 and P-301 as subbase course is not recommended where frost penetration into the subbase is anticipated. Any material suitable for use as base course can also be used on subbase if economy and practicality dictate.

b. Sandwich Construction. Pavements should not be configured such that a pervious granular layer is located between two impervious layers. This type of section is often called "sandwich" construction. Problems are often encountered in "sandwich" construction when water becomes trapped in the granular layer causing a dramatic loss of strength and results in poor performance.

313. SUBGRADE. The subgrade soils are subjected to lower stresses than the surface, base, and subbase courses. Subgrade stresses attenuate with depth, and the controlling subgrade stress is usually at the top of the subgrade, unless unusual conditions exist. Unusual conditions such as a layered subgrade or sharply varying water contents or densities can change the location of the controlling stress. The ability of a particular soil to resist shear and deformation vary with its density and moisture content. Such unusual conditions should be revealed during the soils investigation. Specification Item P-152, Excavation and Embankment, covers the construction and density control of subgrade soils. Table 3-2 shows depths below the subgrade surface to which compaction controls apply.

a. Contamination. A loss of structural capacity can result from contamination of base or subbase elements with fines from underlying subgrade soils. This contamination occurs during pavement construction and during pavement loading. Aggregate contamination results in a reduced ability of the aggregate to distribute and reduce stresses applied to the subgrade. Fine grained soils are most likely to contaminate pavement aggregate. This process is not limited to soft subgrade conditions. Problematic soils may be cohesive or noncohesive and usually exhibit poor drainage properties. Chemical and mechanical stabilization of the subbase or subgrade can be effectively used to reduce aggregate contamination (refer to Section 207). Geotextiles have been found to be effective at providing separation between fine-grained soils and overlying pavement aggregates (FHWA-90-001)(see Appendix 4). In this application, the geotextile is not considered to act as a structural element within the pavement. For separation applications the geotextile is designed based on survivability properties. Refer to FHWA-90-001 (see Appendix 4) for additional information regarding design and construction using separation geotextiles.

b. Example. An apron extension is to be built to accommodate a 340,000-pound (154 000 kg) dual tandem geared aircraft, a soils investigation has shown the subgrade will be noncohesive. In-place densities of the soils have been determined at even foot increments below the ground surface. Design calculations indicate that the top of subgrade in this area will be approximately 10 inches (0.3 m) below the existing grade. Depths and densities may be tabulated as follows:

Depth Below Existing Grade	Depth Below Finished Grade	In-Place Density
1' (0.3 m)	2" (50 mm)	70%
2' (0.6 m)	14" (0.36 m)	84%
3' (0.9 m)	26" (0.66 m)	86%
4' (1.2 m)	38" (0.97 m)	90%
5' (1.5 m)	50" (1.27 m)	93%

Using Table 3-2 values for non-cohesive soils and applying linear interpolation the compaction requirements are as follows:

100%	95%	90%	85%
0-21	21-37	37-52	52-68

Comparison of the tabulations show that for this example in-place density is satisfactory at a depth of 38 inches (0.97 m), being 90 percent within the required 90 percent zone. It will be necessary to compact an additional 1 inch (0.03 m) at 95 percent, and the top 21 inches (0.53 m) of subgrade at 100 percent density.

TABLE 3-2. SUBGRADE COMPACTION REQUIREMENTS FOR FLEXIBLE PAVEMENTS

DESIGN AIRCRAFT	Gross Weight lbs.	NON-COHESIVE SOILS Depth of Compaction In.				COHESIVE SOILS Depth of Compaction In.			
		100%	95%	90%	85%	95%	90%	85%	80%
Single Wheel	30,000	8	8-18	18-32	32-44	6	6-9	9-12	12-17
	50,000	10	10-24	24-36	36-48	6	6-9	9-16	16-20
	75,000	12	12-30	30-40	40-52	6	6-12	12-19	19-25
Dual Wheel (incls. C-130)	50,000	12	12-28	28-38	38-50	6	6-10	10-17	17-22
	100,000	17	17-30	30-42	42-55	6	6-12	12-19	19-25
	150,000	19	19-32	32-46	46-60	7	7-14	14-21	21-28
	200,000	21	21-37	37-53	53-69	9	8-16	16-24	24-32
Dual Tand. (incls. 757, 767, A-300)	100,000	14	14-26	26-38	38-49	6	6-10	10-17	17-22
	200,000	17	17-30	30-43	43-56	6	6-12	12-18	18-26
	300,000	20	20-34	34-48	48-63	7	7-14	14-22	22-29
	400,000	23	23-41	41-59	59-76	9	9-18	18-27	27-36
DC-10	400,000	21	21-36	36-55	55-70	8	8-15	15-20	20-28
L1011	600,000	23	23-41	41-59	59-76	9	9-18	18-27	27-36
747	800,000	23	23-41	41-59	59-76	9	9-18	18-27	27-36

Notes:

1. Noncohesive soils, for the purpose of determining compaction control, are those with a plasticity index (P.I.) of less than 6.
2. Tabulated values denote depths below the finished subgrade above which densities should equal or exceed the indicated percentage of the maximum dry density as specified in Item P-152.
3. The subgrade in cut areas should have natural densities shown or should (a) be compacted from the surface to achieve the required densities, (b) be removed and replaced at the densities shown, or (c) when economics and grades permit, be covered with sufficient select or subbase material so that the uncompacted subgrade is at a depth where the in-place densities are satisfactory.
4. For intermediate aircraft weights use linear interpolation.
5. For swelling soils refer to paragraph 314.
6. 1 inch = 25.4 mm
   1 lb. = 0.454 kg

314. SWELLING SOILS. Swelling soils are clayey soils which exhibit significant volume changes brought on by moisture variations. The potential for volumetric change of a soil due to moisture variation is a function of the type of soil and the likelihood of for moisture fluctuation. Airport pavements constructed on these soils are subject to differential movements causing surface roughness and cracking. The design of pavements in areas of swelling soils should incorporate methods that prevent or reduce the effects of soil volume changes.

a. Soil Type. Only clayey soils containing a significant amount of particular clay minerals are prone to swelling. The clay minerals which cause swelling are, in descending order of swelling activity, are: smectite, illite, and kaolinite. These soils usually have liquid limits above 40 and plasticity indexes above 25.

b. Identification. Soils which exhibit a swell of greater than 3 percent when tested for the California Bearing Ratio (CBR), ASTM D 1883, require treatment. Experience with soils in certain locales is often used to determine when treatment is required.

c. Treatment. Treatment of swelling soils consist of removal and replacement, stabilization, modified compaction efforts and careful control of compaction moisture. Provisions for adequate drainage is of paramount importance when dealing with swelling soils. Recommended treatments for swelling soils are shown in Table 3-3. Local experience and judgment should be applied in dealing with swelling soils to achieve the best results. Care should be taken to minimize water flow along the contact plane between the stabilized/nonstabilized material.

TABLE 3-3. RECOMMENDED TREATMENT OF SWELLING SOILS

Swell Potential (Based on Experience)	Percent Swell Measured (ASTM D 1883)	Potential for Moisture Fluctuation[1]	Treatment
Low	3-5	Low	Compact soil on wet side of optimum (+2% to +3%) to not greater than 90% of appropriate maximum density. [2]
		High	Stabilize soil to a depth of at least 6 in. (150 mm)
Medium	6-10	Low	Stabilize soil to a depth of at least 12 in. (300 mm)
		High	Stabilize soil to a depth of at least 12 in. (300 mm)
High	Over 10	Low	Stabilize soil to a depth of at least 12 in. (300 mm)
		High	For uniform soils, i.e., redeposited clays, stabilize soil to a depth of at least 36 in. (900 mm) or raise grade to bury swelling soil at least 36 in. (900 mm) below pavement section or remove and replace with non-swelling soil. For variable soil deposits depth of treatment should be increased to 60 in. (1300 mm).

Notes: [1]Potential for moisture fluctuation is a judgmental determination and should consider proximity of water table, likelihood of variations in water table, as well as other sources of moisture, and thickness of the swelling soil layer.

[2]When control of swelling is attempted by compacting on the wet side of optimum and reduced density, the design subgrade strength should be based on the higher moisture content and reduced density.

d. Additional Information. Additional information on identifying and handling swelling soils is presented in FAA Reports No. FAA-RD-76-66, Design and Construction of Airport Pavements on Expansive Soils, by R. Gordon McKeen, dated June 1976 and DOT/FAA/PM-85/15, Validation of Procedures for Pavement Design on Expansive Soils, by R. Gordon McKeen, dated July 1985. See Appendix 4.

315. SELECTION OF DESIGN CBR VALUE. Subgrade soils are usually rather variable and the selection of a design CBR value requires some judgment. As a general rule of thumb the design CBR value should be equal to or less than 85% of all the subgrade CBR values. This corresponds to a design value of one standard deviation below the mean as recommended in Chapter 2. In some cases subgrade soils which are significantly different in strength occur in different layers. In these instances several designs should be examined to determine the most economical pavement section. It may be more economical to remove and replace a weak layer than designing for it. On the other hand, circumstances may be such that designing for the weakest layer is more economical. Local conditions will dictate which approach should be used.

316. DESIGN CURVES. Due to the differences in stress distribution characteristics, separate flexible pavement design curves for several gear configurations have been prepared and are presented in Figures 3-2 through 3-15, inclusive. The thicknesses determined from these design charts are for untreated granular bases and subbases and do not include frost effects or stabilized materials. Frost effects and stabilized materials must be handled separately.

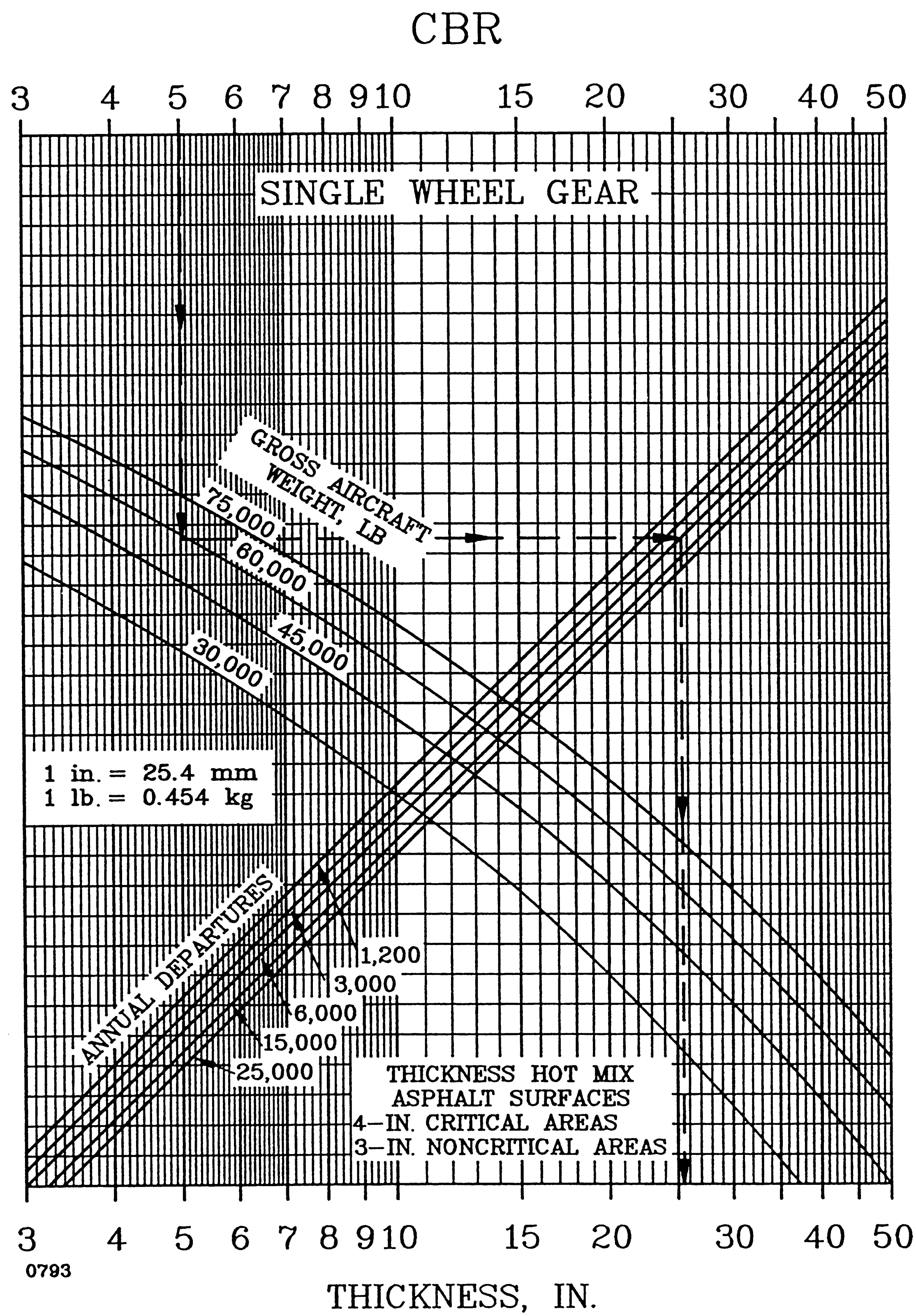


FIGURE 3-2 FLEXIBLE PAVEMENT DESIGN CURVES, SINGLE WHEEL GEAR

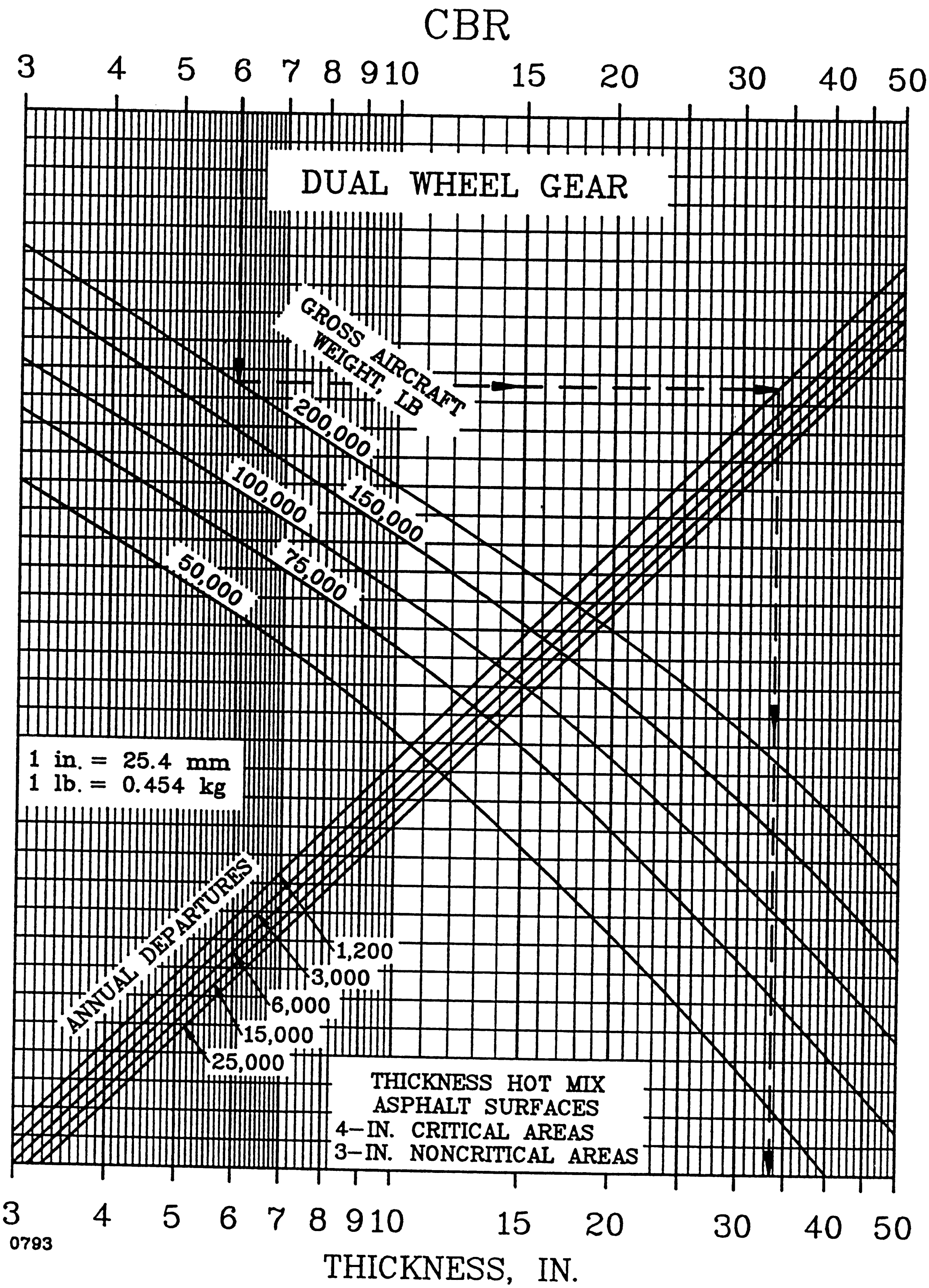


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FIGURE 3-3 FLEXIBLE PAVEMENT DESIGN CURVES, DUAL WHEEL GEAR

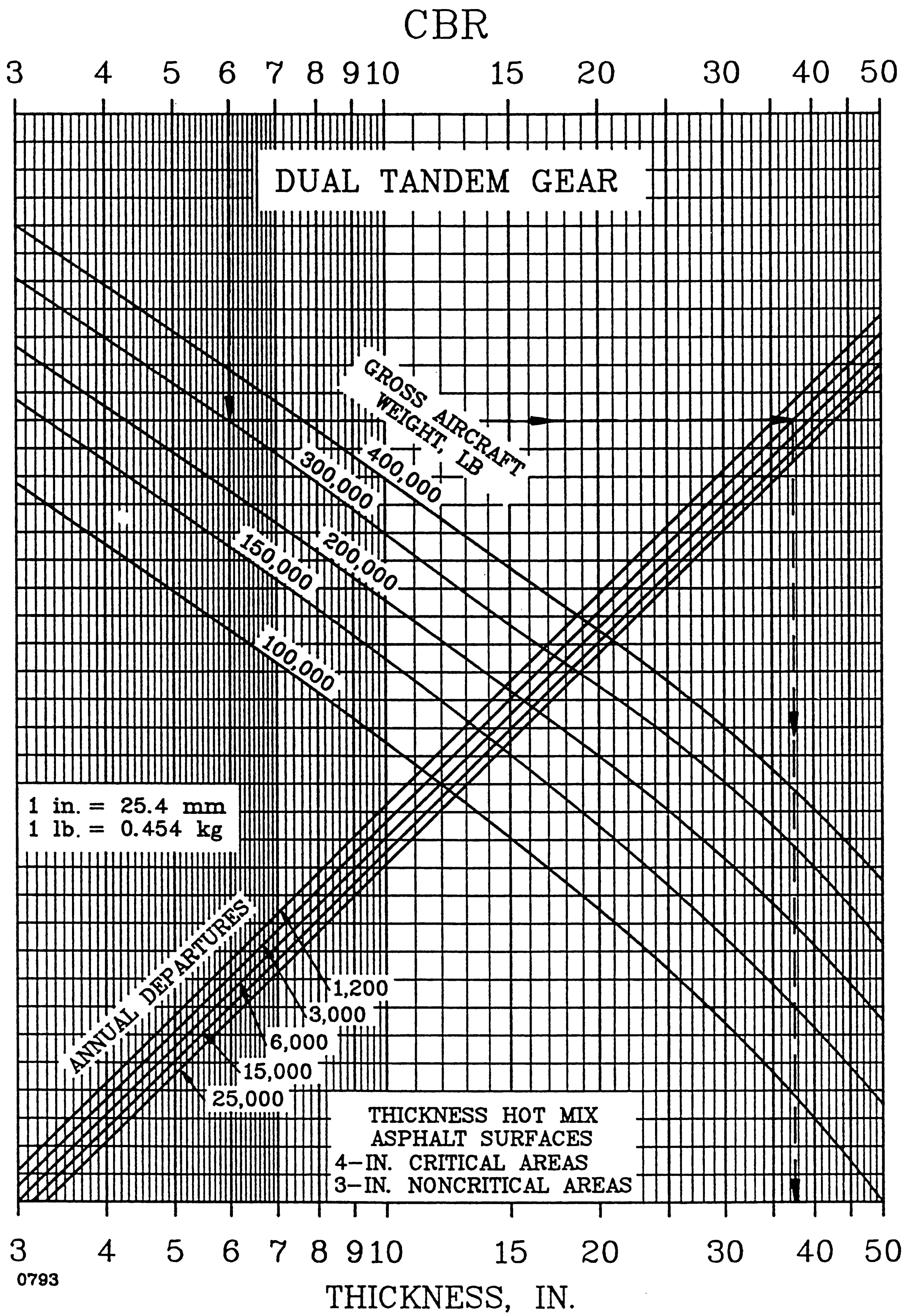


FIGURE 3-4 FLEXIBLE PAVEMENT DESIGN CURVES, DUAL TANDEM GEAR

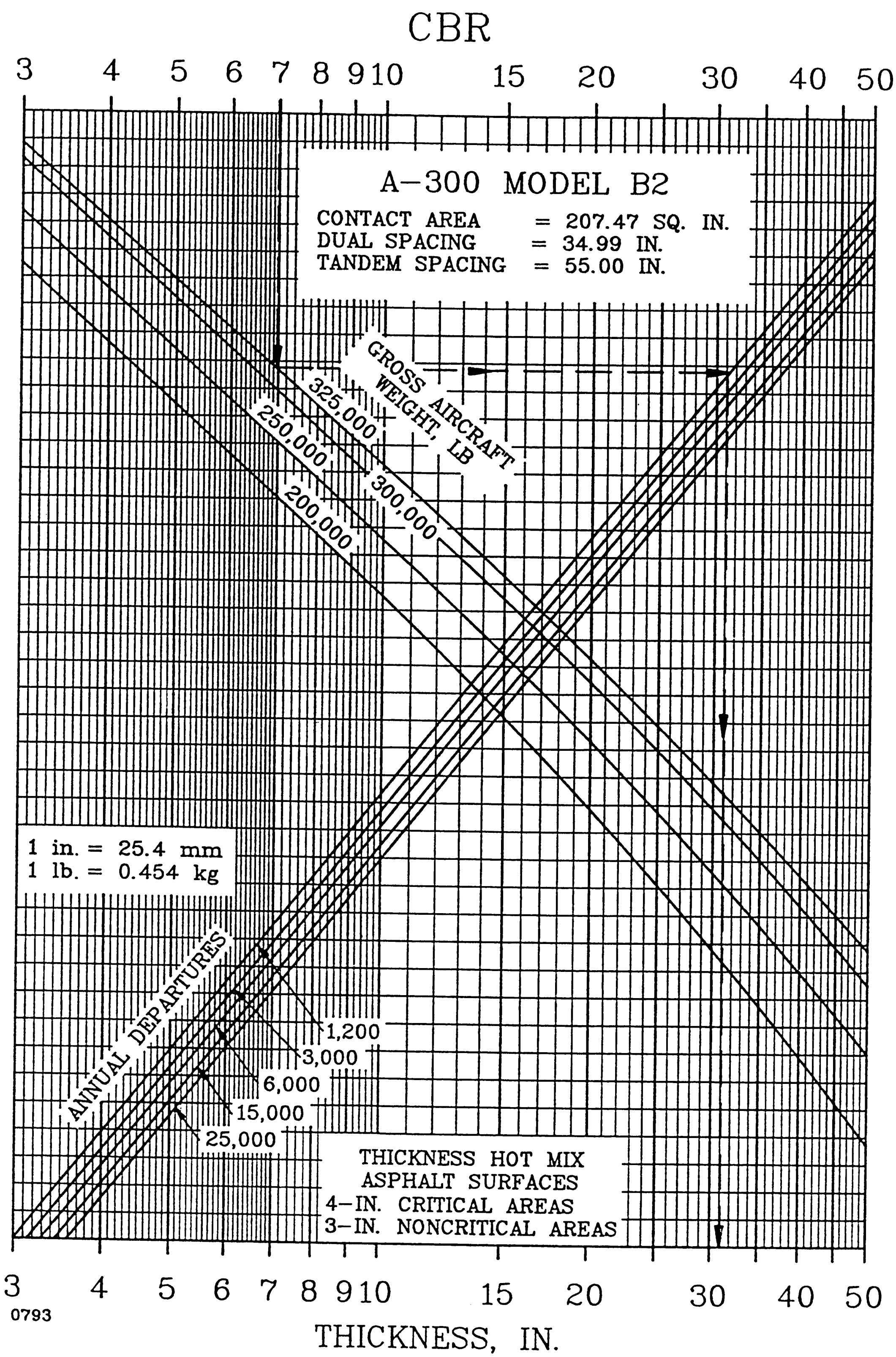


FIGURE 3-5 FLEXIBLE PAVEMENT DESIGN CURVES, A-300 MODEL B2

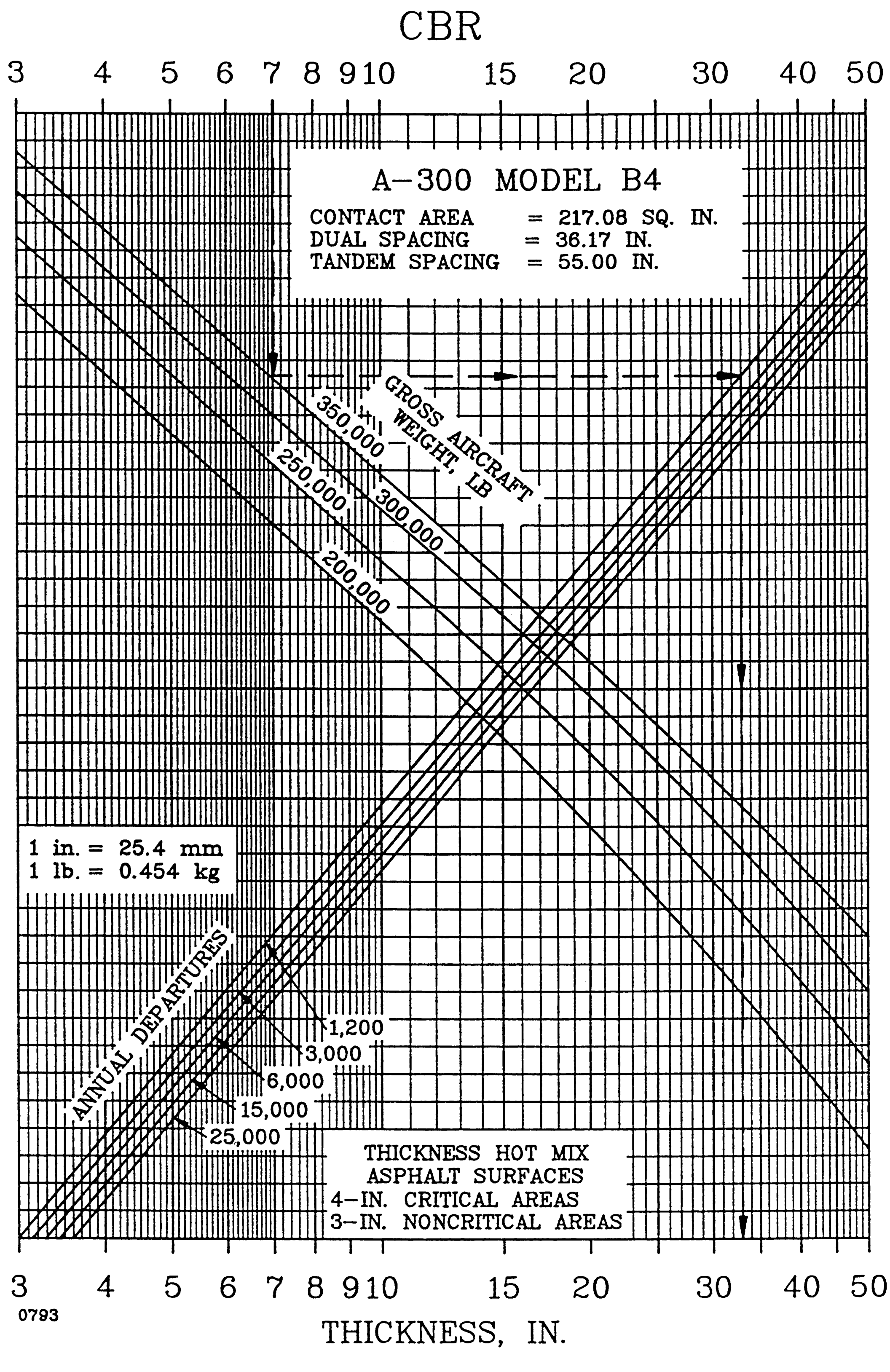


FIGURE 3-6 FLEXIBLE PAVEMENT DESIGN CURVES, A-300 MODEL B4

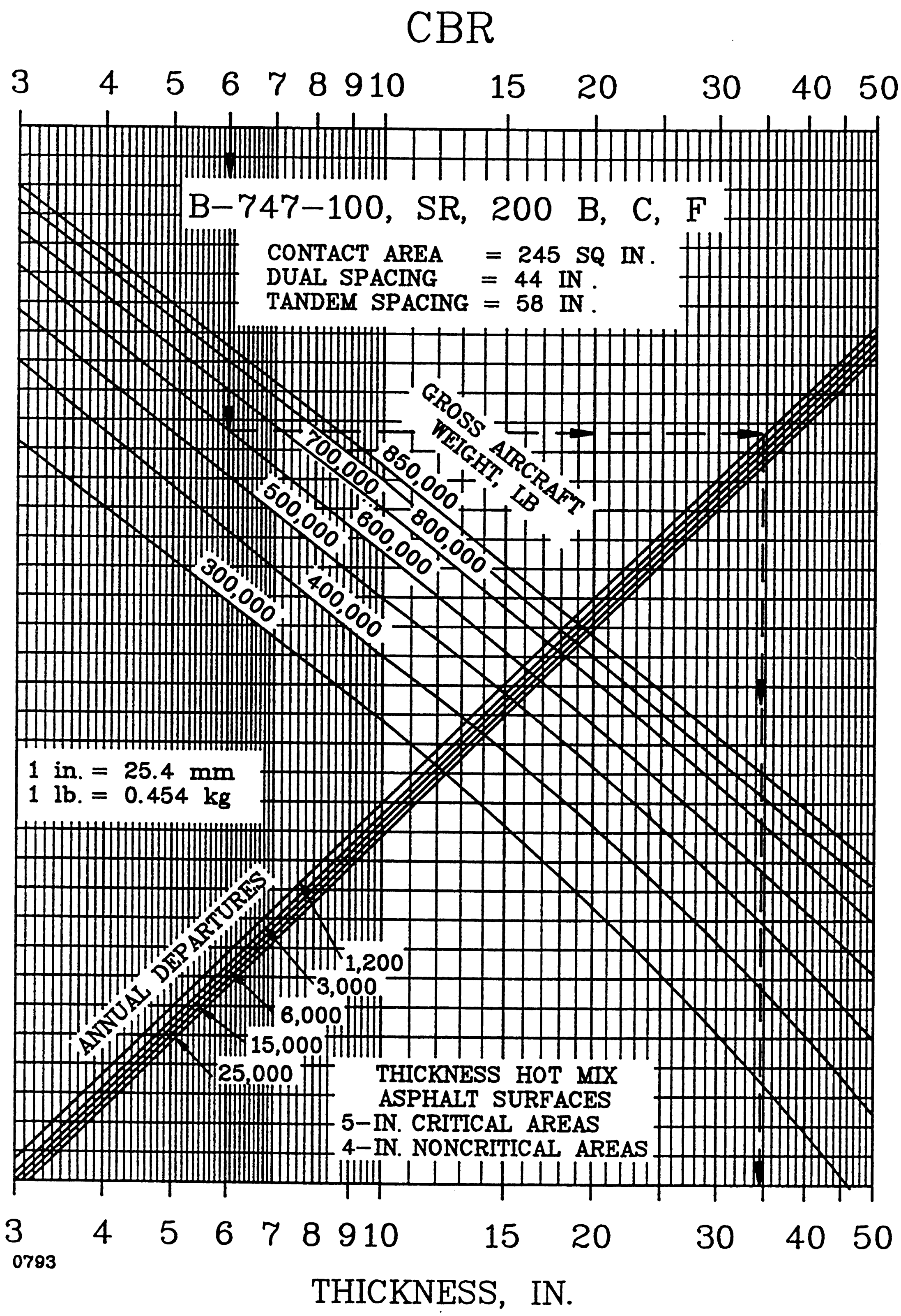


FIGURE 3-7 FLEXIBLE PAVEMENT DESIGN CURVES, B-747-100,SR, 200 B, C, F

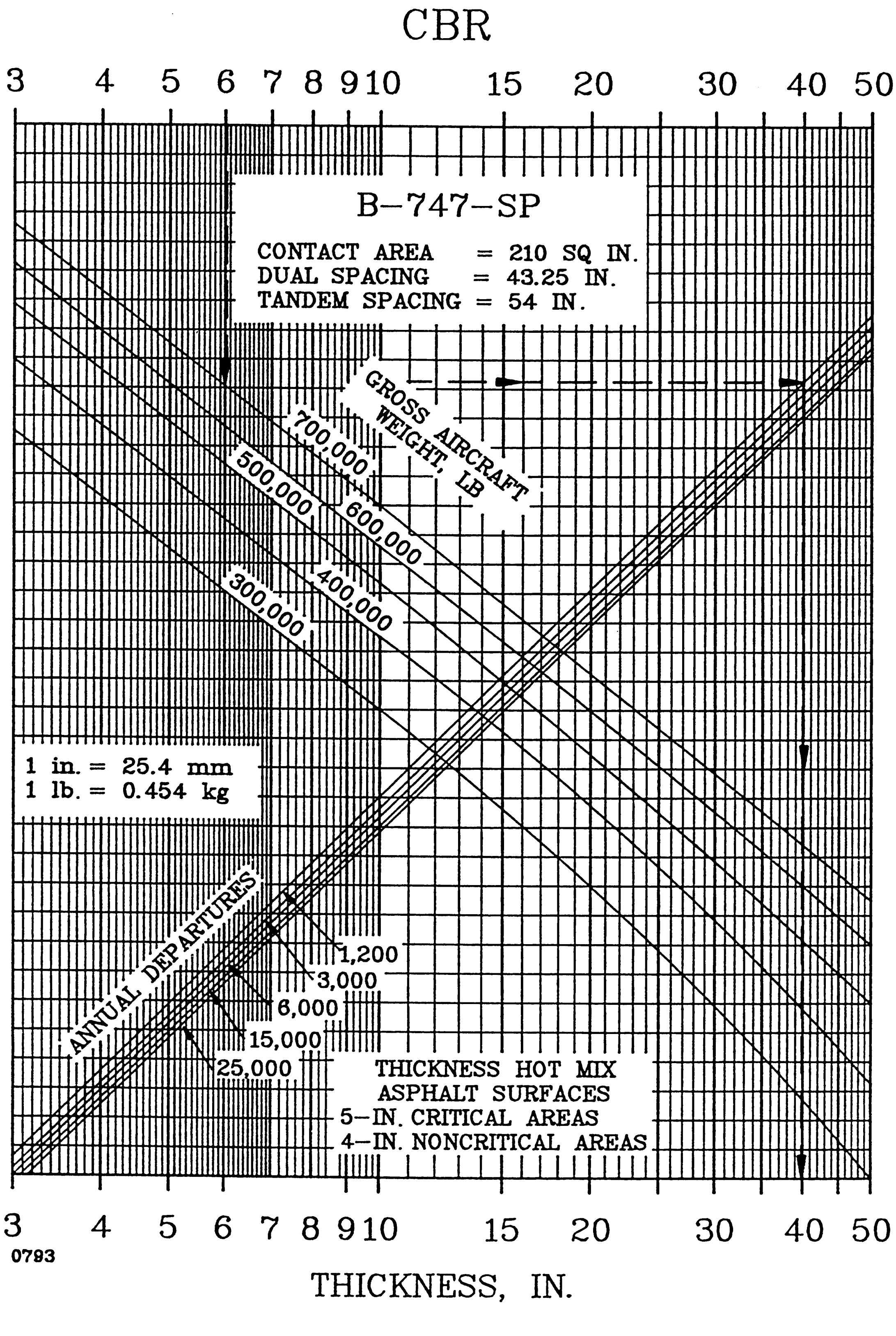


FIGURE 3-8 FLEXIBLE PAVEMENT DESIGN CURVES, B-747-SP

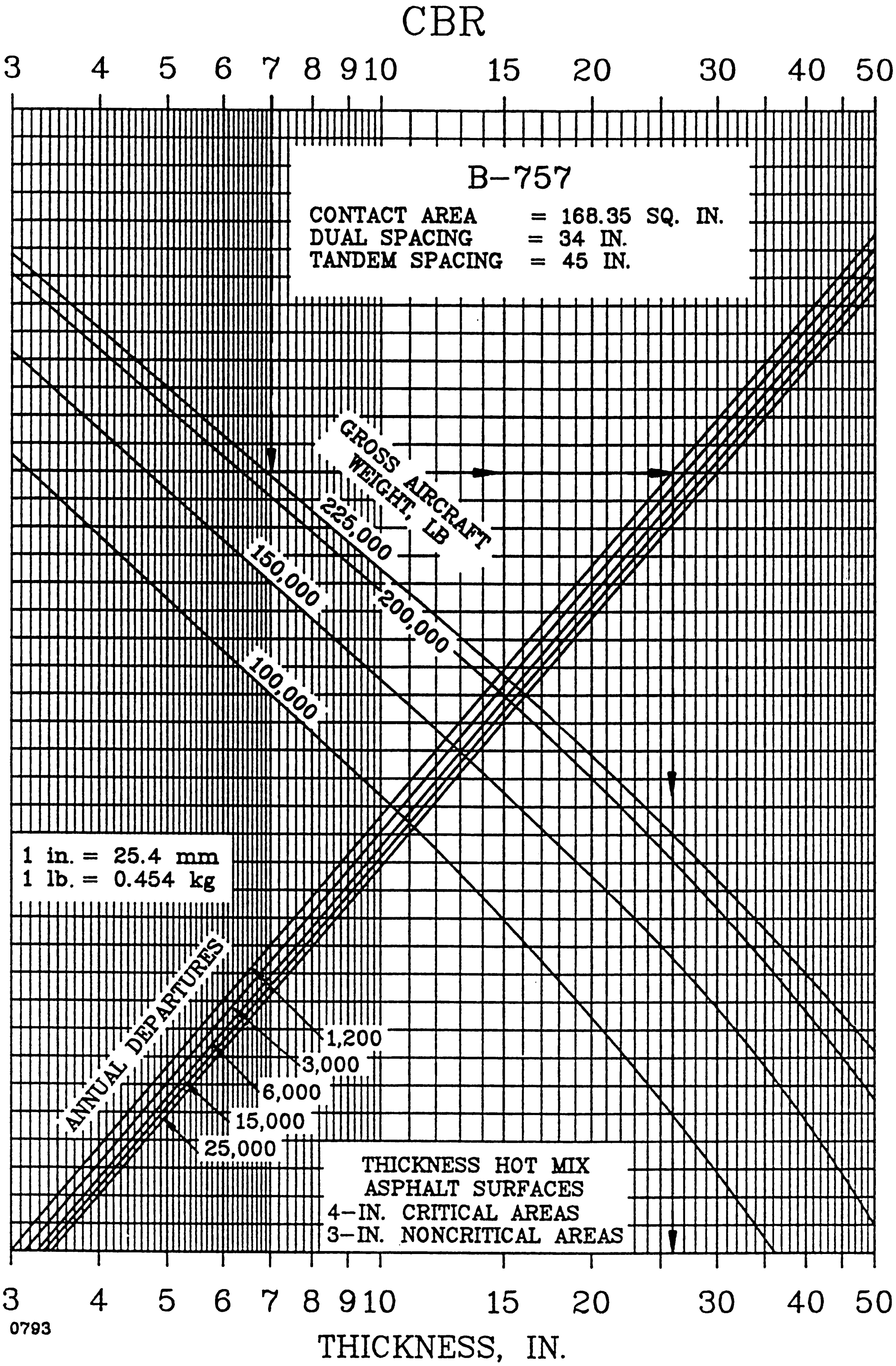


FIGURE 3-9 FLEXIBLE PAVEMENT DESIGN CURVES, B-757

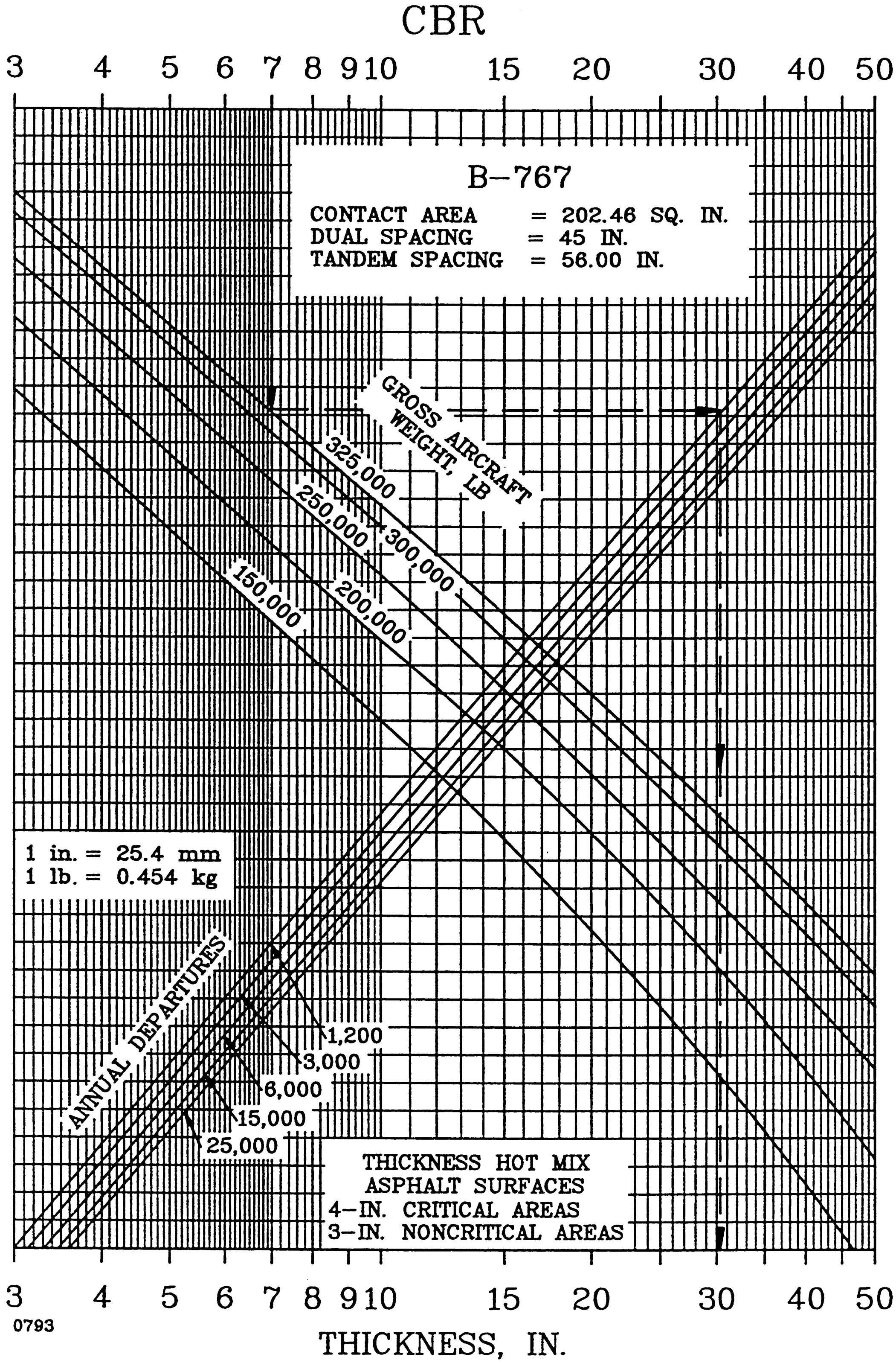


FIGURE 3-10 FLEXIBLE PAVEMENT DESIGN CURVES, B-767

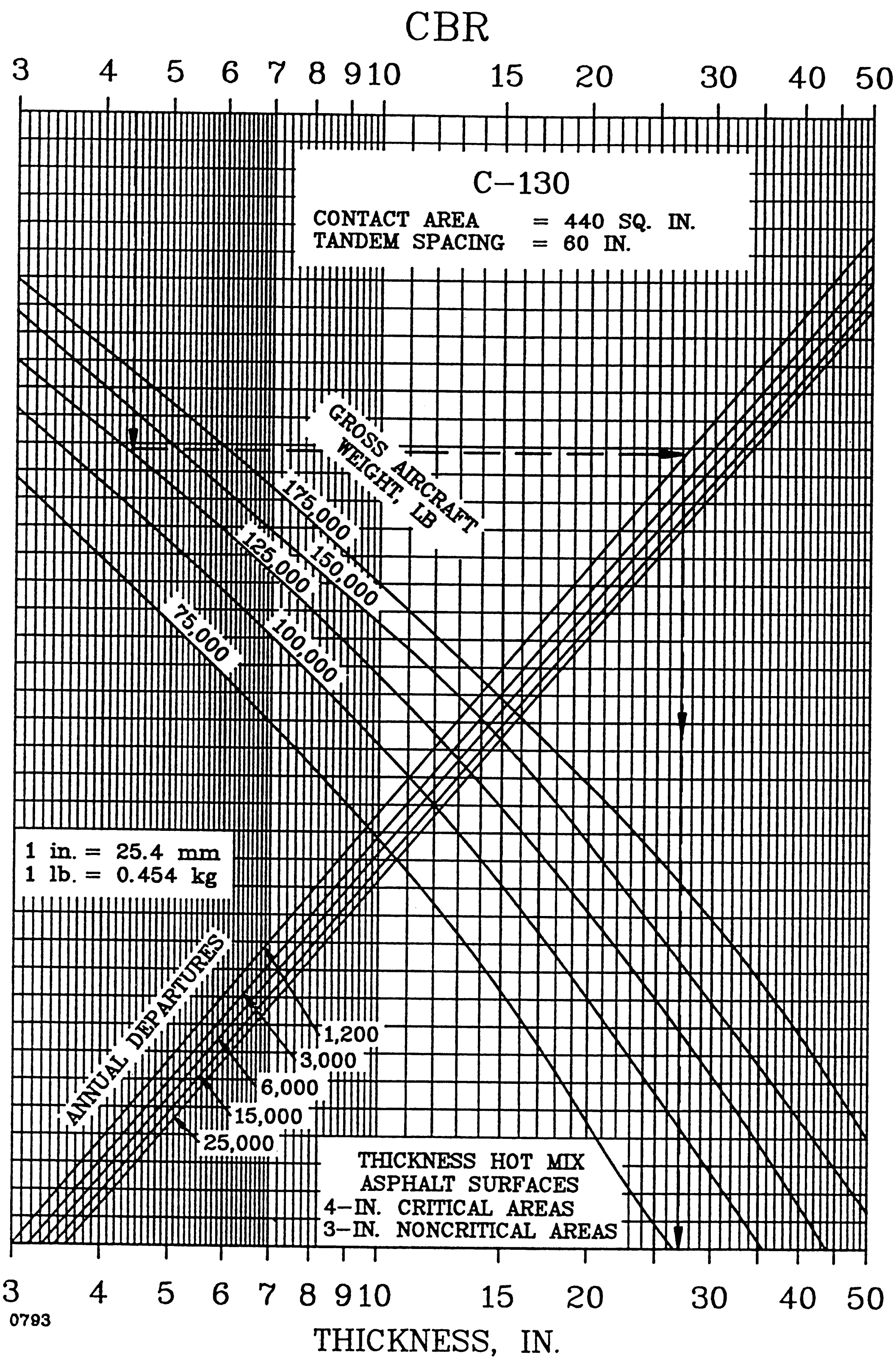


FIGURE 3-11 FLEXIBLE PAVEMENT DESIGN CURVES, C-130

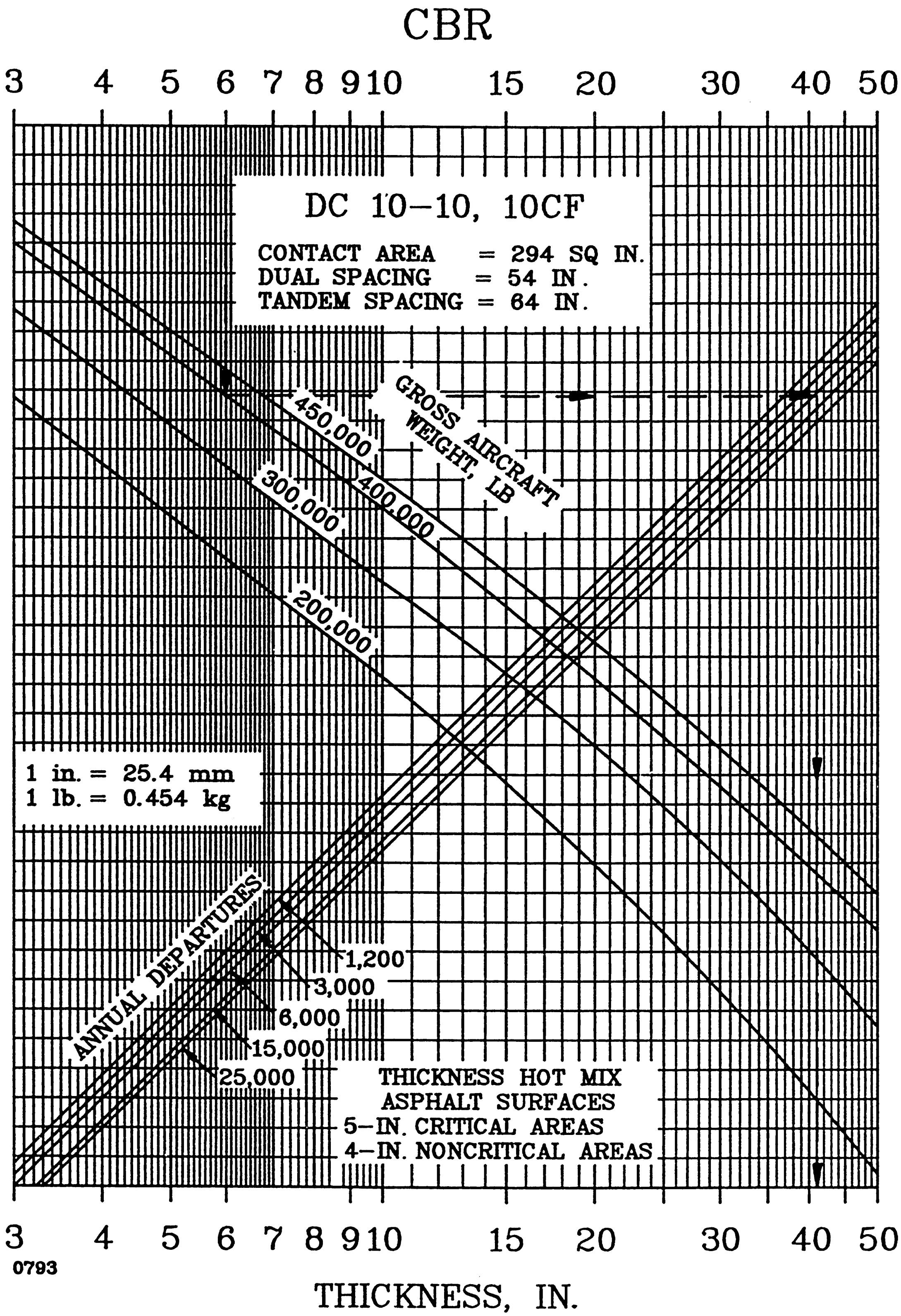


FIGURE 3-12 FLEXIBLE PAVEMENT DESIGN CURVES, DC 10-10, 10CF

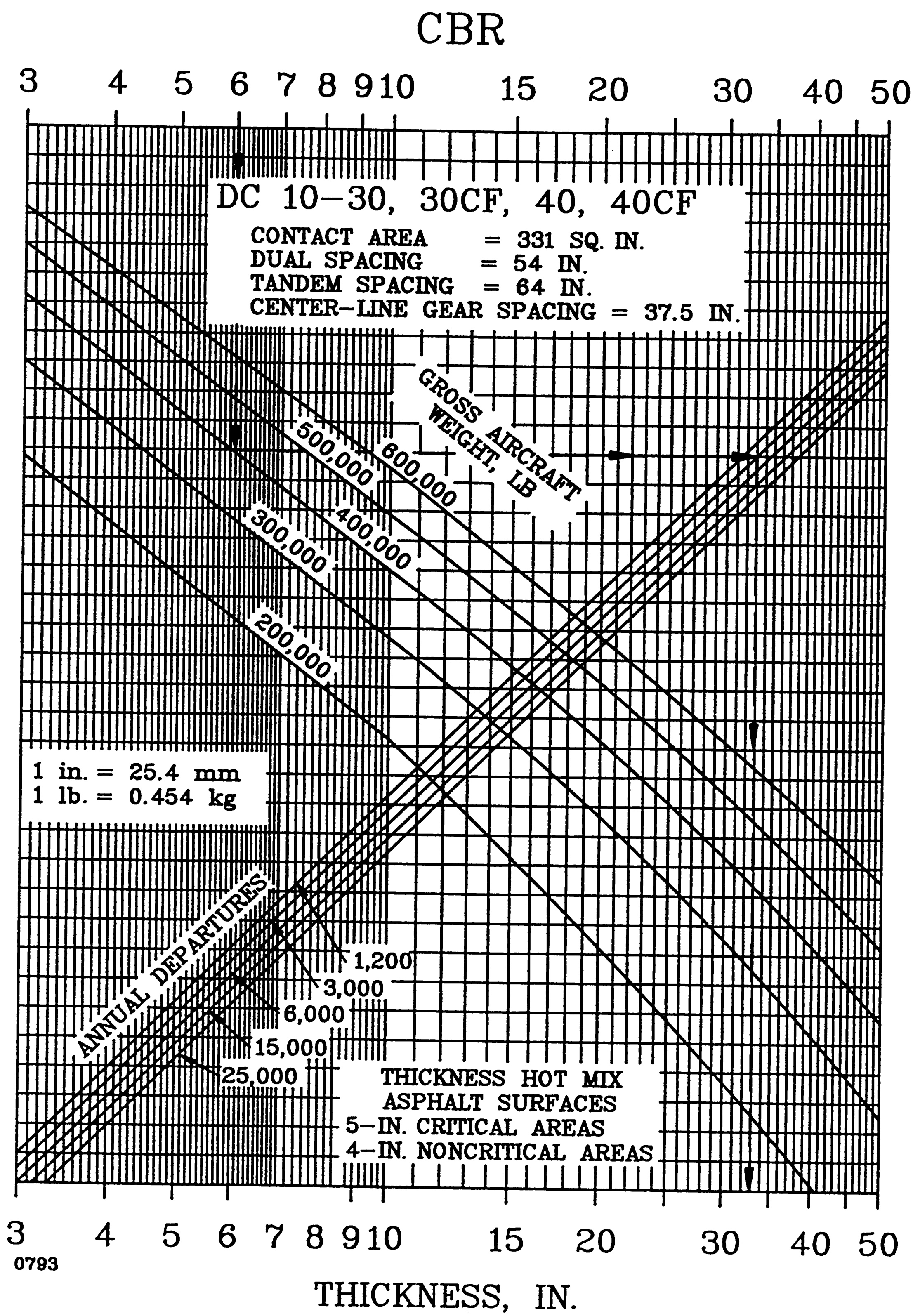


FIGURE 3-13 FLEXIBLE PAVEMENT DESIGN CURVES, DC 10-30, 30CF, 40, 40CF

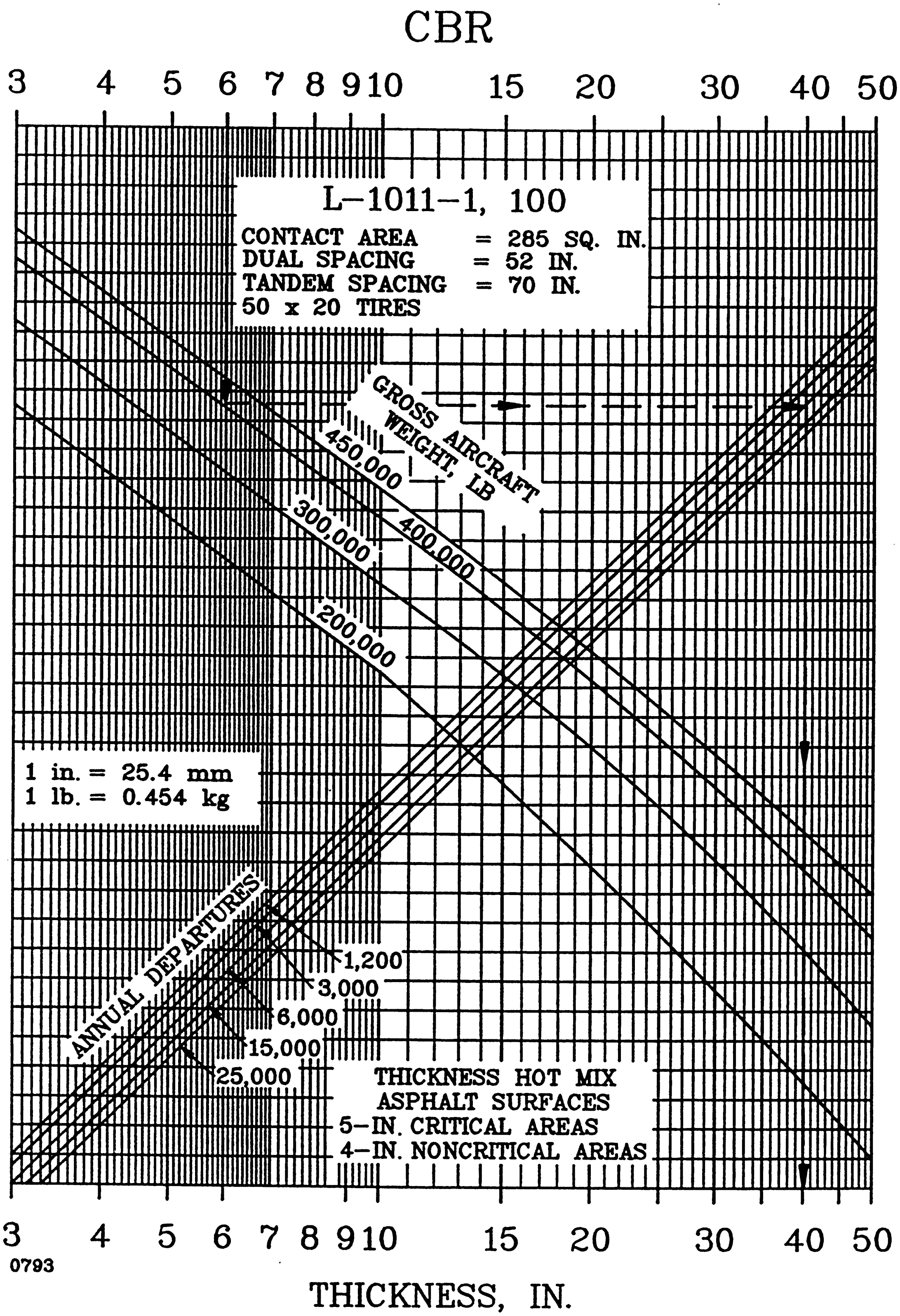


FIGURE 3-14 FLEXIBLE PAVEMENT DESIGN CURVES, L-1011-1, 100

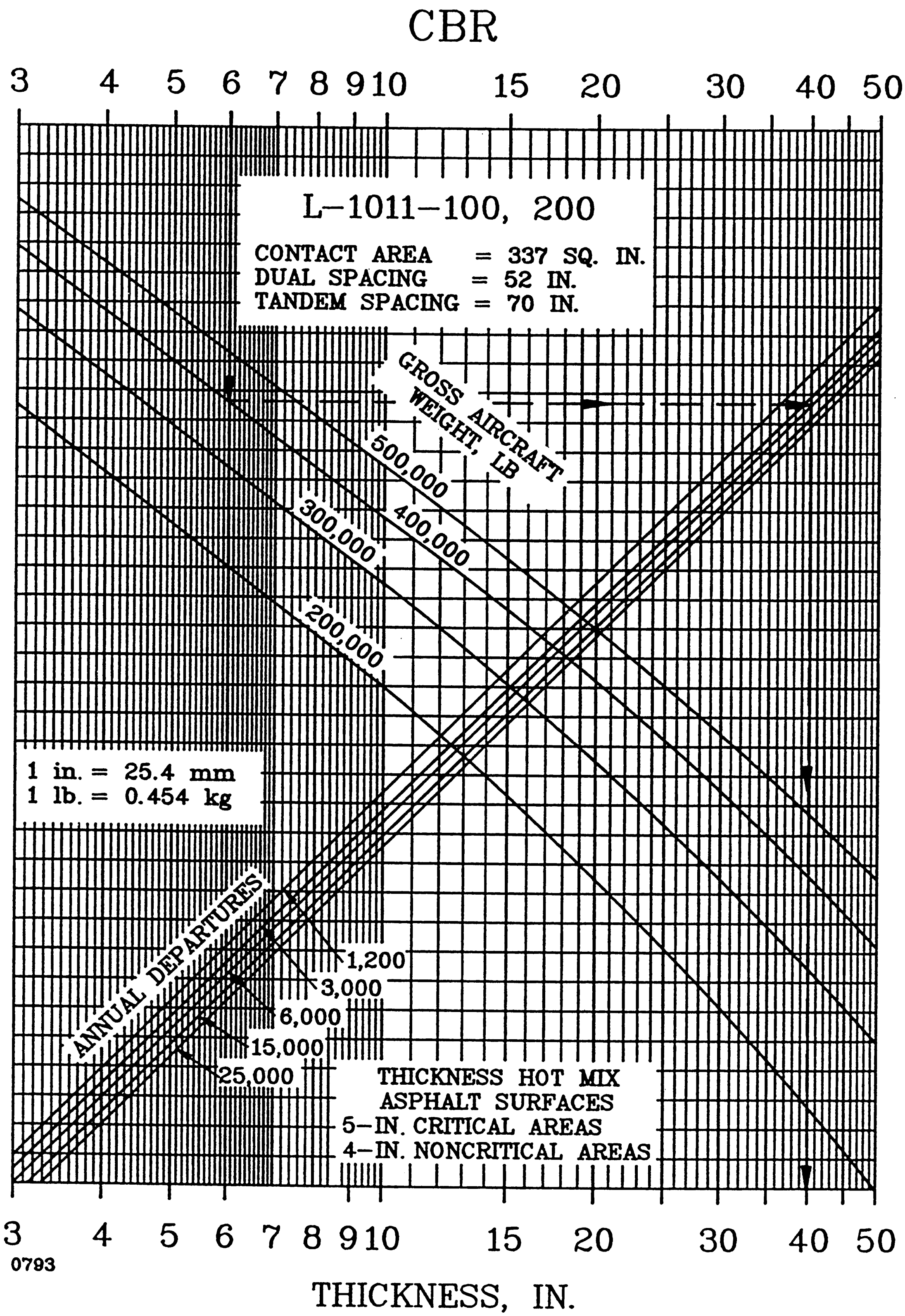


FIGURE 3-15 FLEXIBLE PAVEMENT DESIGN CURVES, L-1011, -100, 200

317. DESIGN INPUTS. Use of the design curves for flexible pavements requires a CBR value for the subgrade material, a CBR value for the subbase material, the gross weight of the design aircraft, and the number of annual departures of the design aircraft. The design curves presented in Figures 3-2 through 3-15 indicate the total pavement thickness required and the thickness of hot mix asphalt surfacing. Table 3-4 gives the minimum thicknesses of base course for various materials and design loadings. For annual departures in excess of 25,000 the total pavement thickness should be increased in accordance with Table 3-5. 1-inch (25 mm) of the thickness increase should be hot mix asphalt surfacing; the remaining thickness increase should be proportioned between base and subbase.

TABLE 3-4. MINIMUM BASE COURSE THICKNESS

Design Aircraft	Design Load Range				Minimum Base Course Thickness	
	lbs.		(kg)		in.	(mm)
Single Wheel	30,000 -	50,000	(13 600 -	22 700)	4	(100)
	50,000 -	75,000	(22 700 -	34 000)	6	(150)
Dual Wheel	50,000 -	100,000	(22 700 -	45 000)	6	(150)
	100,000 -	200,000	(45 000 -	90 700)	8	(200)
Dual Tandem	100,000 -	250,000	(45 000 -	113 400)	6	(150)
	250,000 -	400,000	(113 400 -	181 000)	8	(200)
757 767	200,000 -	400,000	(90 700 -	181 000)	6	(150)
DC-10 L1011	400,000 -	600,000	(181 000 -	272 000)	8	(200)
B-747	400,000 -	600,000	(181 000 -	272 000)	6	(150)
	600,000 -	850,000	(272 000 -	385 700)	8	(200)
C-130	75,000 -	125,000	(34 000 -	56 700)	4	(100)
	125,000 -	175,000	(56 700 -	79 400)	6	(150)

Note: The calculated base course thicknesses should be compared with the minimum base course thicknesses listed above. The greater thickness, calculated or minimum, should be specified in the design section.

318. CRITICAL AND NONCRITICAL AREAS. The design curves, Figures 3-2 through 3-15, are used to determine the total critical pavement thickness, "T", and the surface course thickness requirements. The 0.9T factor for the noncritical pavement applies to the base and subbase courses; the surface course thickness is as noted on the design curves. For the variable section of the transition section and thinned edge, the reduction applies only to the base course. The 0.7T thickness for base shall be the minimum permitted. The subbase thickness shall be increased or varied to provide positive surface drainage of the subgrade surface. Surface course thicknesses are as shown in Figure 3-1. For fractions of an inch of 0.5 or more, use the next higher whole number; for less than 0.5, use the next lower whole number.

TABLE 3-5. PAVEMENT THICKNESS FOR HIGH DEPARTURE LEVELS

Annual Departure Level	Percent of 25,000 Departure Thickness
50,000	104
100,000	108
150,000	110
200,000	112

Note:
The values given in Table 3-5 are based on extrapolation of research data and observations of in-service pavements. Table 3-5 was developed assuming a logarithmic relationship between percent of thickness and departures.

319. DESIGN EXAMPLE. As an example of the use of the design curves, assume a flexible pavement is to be designed for a dual gear aircraft having a gross weight of 75,000 pounds (34 000 kg) and 6,000 annual equivalent departures of the design aircraft. Design CBR values for the subbase and subgrade are 20 and 6, respectively.

a. Total Pavement Thickness. The total pavement thickness required is determined from Figure 3-3. Enter the upper abscissa with the subgrade CBR value, 6. Project vertically downward to the gross weight of the design aircraft, 75,000 pounds (34 000 kg). At the point of intersection of the vertical projection and the aircraft gross weight, make a horizontal projection to the equivalent annual departures, 6000. From the point of intersection of the horizontal projection and the annual departure level, make a vertical projection down to the lower abscissa and read the total pavement thickness; in this example - 23 inches (584 mm).

b. Thickness of Subbase Course. The thickness of the subbase course is determined in a manner similar to the total pavement thickness. Using Figure 3-3, enter the upper abscissa with the design CBR value for the subbase, 20. The chart is used in the same manner as described in "a" above, i.e., vertical projection to aircraft gross weight, horizontal projection to annual departures, and vertical projection to lower abscissa. In this example the thickness obtained is 9.5 inches (241 mm). This means that the combined thickness of hot mix asphalt surface and base course needed over a 20 CBR subbase is 9.5 inches (241 mm), thus leaving a subbase thickness of 23 - 9.5 = 13.5 inches (343 mm).

c. Thickness of Hot Mix Asphalt Surface. As indicated by the note in Figure 3-3, the thickness of hot mix asphalt surface for critical areas is 4 inches (100 mm) and for noncritical, 3 inches (76 mm).

d. Thickness of Base Course. The thickness of base course can be computed by subtracting the thickness of hot mix asphalt surface from the combined thickness of surface and base determined in "b" above; in this example 9.5 - 4.0 = 5.5 (150 mm) of base course. The thickness of base course thus calculated should be compared with the minimum base course thickness required as shown in Table 3-4. Note that the minimum base course thickness is 6 inches (150 mm) from Table 3-4. Therefore the minimum base course thickness from Table 3-4, 6 inches (152 mm), would control. If the minimum base course thickness from Table 3-4 had been less than the calculated thickness, the calculated thickness would have controlled. Note also that use of Item P-208, Aggregate Base Course, as base course is not permissible since the weight of the design aircraft exceeds 60,000 lbs. (27 000 kg).

e. Thickness of Noncritical Areas. The total pavement thickness for noncritical areas is obtained by taking 0.9 of the critical pavement base and subbase thicknesses plus the required hot mix asphalt surface thickness given on the design charts. For the thinned edge portion of the critical and noncritical pavements, the 0.7T factor applies only to the base course because the subbase should allow for transverse drainage. The transition section and surface course requirements are as noted in Figure 3-1.

f. Summary. The thickness calculated in the above paragraphs should be rounded off to even increments as discussed in paragraph 318. If conditions for detrimental frost action exist, another analysis is required. The final design thicknesses for this example would be as follows:

THICKNESS REQUIREMENTS

	Critical in. (mm)	Non-Critical in. (mm)	Edge in. (mm)
Hot Mix Asphalt Surface (P-209 Base)	4 (100)	3 (75)	2 (50)
Base Course (P-209, or P-211)	6 (200)	5 (125)	4 (100)
Subbase Course (P-154)	14 (355)	13 (330)	10 (255)
Transverse Drainage Course (if needed)	0 (0)	3 (75)	8 (205)

320. STABILIZED BASE AND SUBBASE. Stabilized base and subbase courses are necessary for new pavements designed to accommodate jet aircraft weighing 100,000 pounds (45 350 kg) or more. These stabilized courses may be substituted for granular courses using the equivalency factors discussed in paragraph 322. These equivalency factors are based on research studies which measured pavement performance. See FAA Report No. FAA-RD-73-198, Volumes I, II, and III. Comparative Performance of Structural Layers in Pavement Systems. See Appendix 3. A range of equivalency factors is given because the factor is sensitive to a number of variables such as layer thickness, stabilizing agent type and quantity, location of stabilized layer in the pavement structure, etc. Exceptions to the policy requiring stabilized base and subbase may be made on the basis of superior materials being available, such as 100 percent crushed, hard, closely graded stone. These materials should exhibit a remolded soaked CBR minimum of 100 for base and 35 for subbase. In areas subject to frost penetration, the materials should meet permeability and nonfrost susceptibility tests in addition to the CBR requirements. Other exceptions to the policy requiring stabilized base and subbase should be based on proven performance of a granular material such as lime rock in the State of Florida. Proven performance in this instance means a history of satisfactory airport pavements using the materials. This history of satisfactory performance should be under aircraft loadings and climatic conditions comparable to those anticipated.

321. SUBBASE AND BASE EQUIVALENCY FACTORS. It is sometimes advantageous to substitute higher quality materials for subbase and base course than the standard FAA subbase and base material. The structural benefits of using a higher quality material is expressed in the form of equivalency factors. Equivalency factors indicate the substitution thickness ratios applicable to various higher quality layers. Stabilized subbase and base courses are designed in this way. Note that substitution of lesser quality materials for higher quality materials, regardless of thickness, is not permitted. The designer is reminded that even though structural considerations for flexible pavements with high quality subbase and base may result in thinner flexible pavements; frost effects must still be considered and could require thicknesses greater than the thickness for structural considerations.

a. Minimum Total Pavement Thickness. The minimum total pavement thickness calculated, after all substitutions and equivalencies have been made, should not be less than the total pavement thickness required by a 20 CBR subgrade on the appropriate design curve.

b. Granular Subbase. The FAA standard for granular subbase is Item P-154, Subbase Course. In some instances it may be advantageous to utilize nonstabilized granular material of higher quality than P-154 as subbase course. Since these materials possess higher strength than P-154, equivalency factor ranges are established whereby a lesser thickness of high quality granular may be used in lieu of the required thickness of P-154. In developing the equivalency factors the standard granular subbase course, P-154, was used as the basis. Thicknesses computed from the design curves assume P-154 will be used as the subbase. If a granular material of higher quality is substituted for Item P-154, the thickness of the higher quality layer should be less than P-154. The lesser thickness is computed by dividing the required thickness of granular subbase, P-154, by the appropriate equivalency factor. In establishing the equivalency factors the CBR of the standard granular subbase, P-154, was assumed to be 20. The equivalency factor ranges are given below in Table 3-6:

TABLE 3-6. RECOMMENDED EQUIVALENCY FACTOR RANGES FOR HIGH QUALITY GRANULAR SUBBASE

Material	Equivalency Factor Range
P-208, Aggregate Base Course	1.0 - 1.5
P-209, Crushed Aggregate Base Course	1.2 - 1.8
P-211, Lime Rock Base Course	1.0 - 1.5

c. **Stabilized Subbase.** Stabilized subbases also offer considerably higher strength to the pavement than P-154. Recommended equivalency factors associated with stabilized subbase are presented in Table 3-7.

TABLE 3-7. RECOMMENDED EQUIVALENCY FACTOR RANGES FOR STABILIZED SUBBASE

Material	Equivalency Factor Range
P-301, Soil Cement Base Course	1.0 - 1.5
P-304, Cement Treated Base Course	1.6 - 2.3
P-306, Econocrete Subbase Course	1.6 - 2.3
P-401, Plant Mix Bituminous Pavements	1.7 - 2.3

d. **Granular Base.** The FAA standard for granular base is Item P-209, Crushed Aggregate Base Course. In some instances it may be advantageous to utilize other nonstabilized granular material as base course. Other materials acceptable for use as granular base course are as follows:

TABLE 3-8. RECOMMENDED EQUIVALENCY FACTOR RANGES FOR GRANULAR BASE

Material	Equivalency Factor Range
P-208, Aggregate Base Course	1.0[1]
P-211, Lime Rock Base Course	1.0

[1]Substitution of P-208 for P-209 is permissible only if the gross weight of the design aircraft is 60,000 lbs (27 000 kg) or less. In addition, if P-208 is substituted for P-209, the required thickness of hot mix asphalt surfacing shown on the design curves should be increased 1 inch (25 mm).

e. **Stabilized Base.** Stabilized base courses offer structural benefits to a flexible pavement in much the same manner as stabilized subbase. The benefits are expressed as equivalency factors similar to those shown for stabilized subbase. In developing the equivalency factors Item P-209, Crushed Aggregate Base Course, with an assumed CBR of 80 was used as the basis for comparison. The thickness of stabilized base is computed by dividing the granular base course thickness requirement by the appropriate equivalency factor. The equivalency factor ranges are given below in Table 3-9. Ranges of equivalency factors are shown rather than single values since variations in the quality of materials, construction techniques, and control can influence the equivalency factor. In the selection of equivalency factors, consideration should be given to the traffic using the pavement, total pavement thickness, and the thickness of the individual layer. For example, a thin layer in a pavement structure subjected to heavy loads spread over large areas will result in an equivalency factor near the low end of the range. Conversely, light loads on thick layers will call for equivalency factors near the upper end of the ranges.

TABLE 3-9. RECOMMENDED EQUIVALENCY FACTOR RANGES FOR STABILIZED BASE

Material	Equivalency Factor Range
P-304, Cement Treated Base Course	1.2 - 1.6
P-306, Econocrete Subbase Course	1.2 - 1.6
P-401, Plant Mix Bituminous Pavements	1.2 - 1.6

Note: Reflection cracking may be encountered when P-304 or P-306 is used as base for a flexible pavement. The thickness of the hot mix asphalt surfacing course should be at least 4 inches (100 mm) to minimize reflection cracking in these instances.

f. **Example.** As an example of the use of equivalency factors, assume a flexible pavement is required to serve a design aircraft weighing 300,000 pounds (91 000 kg) with a dual tandem gear. The equivalent annual departures are 15,000. The design CBR for the subgrade is 7. Item P-401 will be used for the base course and the subbase course.

(1) **Total Pavement Thickness Unstabilized.** Enter Figure 3-4 with the subgrade CBR value of 7 and read a total pavement thickness of 37.5 inches (953 mm). This thickness includes surfacing, granular base (P-209) and granular subbase (P-154)

(2) **Thickness of Base and Surface Unstabilized.** Re-enter Figure 3-4 with the assumed subbase CBR (P-154) of 20 (see paragraph 321 b.) and read a thickness of 17.0 inches (432 mm). This thickness includes surfacing and granular base (P-209). The note on Figure 3-4 states that the thickness of surfacing for critical areas is 4 inches (100 mm).

(3) **Unstabilized Section.** The unstabilized section would thus consist of 4 inches (100 mm) of surfacing, 13 inches (330 mm) of granular base (P-209) and 20 1/2 inches (520 mm) of granular subbase (P-154).

(4) **Stabilized Base Thickness.** Assume the equivalency factor for P-401 base material to be 1.4. The required thickness of stabilized base is determined by dividing the thickness of granular base calculated in step (3) above by the equivalency factor. In this example 13 inches (330 mm) would be divided by 1.4 yielding 9 inches (230 mm).

(5) **Stabilized Subbase Thickness.** Referring to Table 3-6, assume the equivalency factor for P-401 used as subbase is 2.0. Divide the thickness of granular subbase 20 1/2 inches (520 mm) by 2.0 which yields 10 inches (255 mm) of P-401 subbase.

(6) **Stabilized Section.** The stabilized section would be 4 inches (100 mm) of surfacing, 9 inches (230 mm) of stabilized base (P-401) and 10 inches (255 mm) of stabilized subbase (P-401).

(7) **Check Minimum Thickness.** The total pavement thickness given above 4 + 9 + 10 = 23 inches (585 mm) is then compared to the total pavement thickness required for a CBR of 20. This was done in step (2) above and gave a thickness of 17.0 inches (430 mm). Since the calculated thickness of 23 inches (585 mm) is larger than the CBR=20 minimum thickness of 17 inches (430 mm), the design is adequate. Had the CBR=20 thickness exceeded the calculated thickness, the subbase thickness would have been increased to make up the difference.

322. FULL-DEPTH ASPHALT PAVEMENTS. Full-depth asphalt pavements contain asphaltic cement in all components above the prepared subgrade. The design of full-depth asphalt pavements can be accomplished using the equivalency factors presented in paragraph 321 and illustrated in paragraph 321f. Manual Series No. 11 prepared by the Asphalt Institute, dated January 1973, can also be used to design full-depth asphalt pavements when approved by the FAA.

323. FROST EFFECTS. Frost protection should be provided in areas where conditions conductive to detrimental frost action exist. Levels of frost protection are given in paragraph 308b of this document. Frost considerations may result in thicker subbase courses than the thicknesses needed for structural support.

a. Example. An example of pavement design for seasonal frost follows. Assume the same design conditions as in paragraph 321f above.

(1) **Structural Requirements.** The structural requirements for the example are: 4 inches (100 mm) of surfacing, 9 inches (230 mm) of stabilized base, and 10 inches (255 mm) of stabilized subbase. This section provides a total pavement thickness of 23 inches (585 mm).

(2) **Determine Soil Frost Group.** Assume the subgrade soil is a clayey sand SC with 10% of the material finer than 0.02 mm. The unit dry weight of the subgrade soil is 115 pcf (184 kg/cu m). The soil frost group is found in Table 2-4 and in this example is FG-2.

(3) **Determine the Depth of Frost Penetration.** The design air freezing index for the area is 350 degree days. Referring to figure 2-6 the depth of frost penetration is found to be 28 inches.

(4) **Types of Frost protection.** Several levels of frost protection are possible as follows:

(i) **Complete Frost Protection.** Complete frost protection would require the pavement section be increased from 23 inches (585 mm) to 28 inches (710 mm). This would require placing 5 inches (125 mm) of nonfrost susceptible material beneath the structural section.

(ii) **Limited Frost Protection.** Limited subgrade frost penetration provides nonfrost susceptible material to a depth of 65% of the depth of frost penetration. In this example, 65% of 28 inches (710 mm) equals 18 inches (460 mm). Since the structural design section provides a total pavement thickness of 23 inches (585 mm), no further protection is required. The structural section provides more than enough protection to satisfy the limited subgrade frost penetration requirements.

(iii) **Reduced Subgrade Strength.** The reduced subgrade strength rating for an FG-2 soil is found in paragraph 308a.(3) and is a CBR of 7. Since the design CBR used in the example was 7, the structural design is adequate for the reduced subgrade strength method of frost protection. As has been previously mentioned, this method is intended to provide adequate structural support when the frost is melting.

(5) **Summary.** In summary, for areas sensitive to pavement heave due to frost action the complete protection method should be used. This would add 4 inches (100 mm) of nonfrost susceptible material to the structural section. In areas where some degree of pavement heave due to frost action can be tolerated, the structural section will be adequate. The same is true for providing structural support during periods of frost melting, i.e. the structural section is adequate.

SECTION 3. RIGID PAVEMENT DESIGN

324. GENERAL. Rigid pavements for airports are composed of portland cement concrete placed on a granular or treated subbase course that is supported on a compacted subgrade. Under certain conditions, a subbase is not required, see paragraph 326.

325. CONCRETE PAVEMENT. The concrete surface must provide a nonskid surface, prevent the infiltration of surface water, and provide structural support to the Item P-501, Cement Concrete Pavement.

326. SUBBASE. The purpose of a subbase under a rigid pavement is to provide uniform stable support for the pavement slabs. A minimum thickness of 4 inches (100 mm) of subbase is required under all rigid pavements, except as shown in Table 3-10 below:

TABLE 3-10. CONDITIONS WHERE NO SUBBASE IS REQUIRED

Soil Classification	Good Drainage		Poor Drainage	
	No Frost	Frost	No Frost	Frost
GW	X	X	X	X
GP	X	X	X	
GM	X			
GC	X			
SW	X			

Note: X indicates conditions where no subbase is required.

327. SUBBASE QUALITY. The standard FAA subbase for rigid pavements is 4 inches (100 mm) of Item P-154, Subbase Course. In some instances it may be desirable to use higher quality materials or thicknesses of P-154 greater than 4 inches (100 mm). The following materials are acceptable for use as subbase under rigid pavements:

Item P-154 - Subbase Course
Item P-208 - Aggregate Base Course
Item P-209 - Crushed Aggregate Base Course
Item P-211 - Lime Rock Base Course
Item P-304 - Cement Treated Base Course
Item P-306 - Econocrete Subbase Course
Item P-401 - Plant Mix Bituminous Pavements

Materials of higher quality than P-154 and/or greater thicknesses of subbase are considered in the design process through the foundation modulus (k value). The costs of providing the additional thickness or higher quality subbase should be weighed against the savings in concrete thickness.

328. STABILIZED SUBBASE. Stabilized subbase is required for all new rigid pavements designed to accommodate aircraft weighing 100,000 pounds (45 400 kg) or more. Stabilized subbases are as follows:

Item P-304 - Cement Treated Base Course
Item P-306 - Econocrete Subbase Course
Item P-401 - Plant Mix Bituminous Pavements

The structural benefit imparted to a pavement section by a stabilized subbase is reflected in the modulus of subgrade reaction assigned to the foundation. Exceptions to the policy of using stabilized subbase are the same as given in paragraph 320.

329. SUBGRADE. The subgrade materials under a rigid pavement should be compacted to provide adequate stability and uniform support as with flexible pavement; however, the compaction requirements for rigid pavements are not as stringent as flexible pavement due to the relatively lower subgrade stress. For cohesive soils used in fill sections, the entire fill shall be compacted to 90 percent maximum density. For cohesive soils in cut sections, the top 6 inches (150 mm) of the subgrade shall be compacted to 90 percent maximum density. For noncohesive soils used in fill sections, the top 6 inches (150 mm) of fill shall be compacted to 100 percent maximum density, and the remainder of the fill shall be compacted to 95 percent maximum density. For cut sections in noncohesive soils, the top 6 inches (150 mm) of subgrade shall be compacted to 100 percent maximum density and the next 18 inches (460 mm) of subgrade shall be compacted to 95 percent maximum density. Swelling soils will require special considerations. Paragraph 314 contains guidance on the identification and treatment of swelling soils.

a. Contamination. In rigid pavement systems, repeated loading may cause intermixing of soft subgrade soils and aggregate base or subbase. This mixing may create voids below the pavement in which moisture can accumulate causing a pumping situation to occur. Chemical and mechanical stabilization of the subbase or subgrade can be effectively used to reduce aggregate contamination (refer to Section 207). Geotextiles have been found to be effective at providing separation between fine-grained subgrade soils and pavement aggregates (FHWA-90-001) (see Appendix 4). Geotextiles should be considered for separation between fine-grained soils and overlying pavement aggregates. In this application, the geotextile is not considered to act as a structural element within the pavement. Therefore, the modulus of the base or subbase is not considered to be increased when a geotextile is used for stabilization. For separation applications, the geotextile is designed based on survivability properties. Refer to FHWA-90-001 (see Appendix 4) for additional information regarding design and construction using separation geotextiles.

330. DETERMINATION OF FOUNDATION MODULUS (k VALUE) FOR RIGID PAVEMENT. In addition to the soils survey and analysis and classification of subgrade conditions, the determination of the foundation modulus is required for rigid pavement design. The foundation modulus (k value) should be assigned to the material directly beneath the concrete pavement. However, it is recommended that a k value be established for the subgrade and then corrected to account for the effects of subbase.

a. Determination of k Value for Subgrade. The preferred method of determining the subgrade modulus is by testing a limited section of embankment which has been constructed to the required specifications. The plate bearing test procedures are given in AASHTO T 222, Nonrepetitive Static Plate Load Test of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements. If the construction and testing of a test section of embankment is impractical, the values listed in Table 2-3 may be used. The designer is cautioned that the values in Table 2-3 are approximate and engineering judgment should be used in selecting a design value. Fortunately, rigid pavement is not too sensitive to k value and an error in estimating k will not have a large impact on rigid pavement thickness.

b. Determination of k Value for Granular Subbase. The determination of a foundation modulus on top of a subbase by testing is usually not practical, at least in the design phase. Usually, the embankment and subbase will not be in place in time to perform any field tests. The assignment of a k value will have to be done without the benefit of testing. The probable increase in k value associated with various thicknesses of different subbase materials is shown in Figures 2-5a and 2-5b. Figure 2-5a is intended for use when the subbase is composed of well graded crushed aggregate such as P-209. Figure 2-5b applies to bank-run sand and gravel such as P-154. These curves apply to unstabilized granular materials. Values shown in Figures 2-5a and 2-5b are considered guides and may be tempered by local experience.

c. Determination of k Value for Stabilized Subbase. As with granular subbase, the effect of stabilized subbase is reflected in the foundation modulus. Figure 3-16 shows the probable increase in k value with various thicknesses of stabilized subbase located on subgrades of varying moduli. Figure 3-16 is applicable to cement stabilized (P-304) Econocrete (P-306), and bituminous stabilized (P-401) layers. Figure 3-16 was developed by assuming a stabilized layer is twice as effective as well-graded crushed aggregate in increasing the subgrade modulus. Stabilized layers of lesser quality than P-304, P-306 or P-401 should be assigned somewhat lower k values. After a k value is assigned to the stabilized subbase, the design procedure is the same as described in paragraph 331.

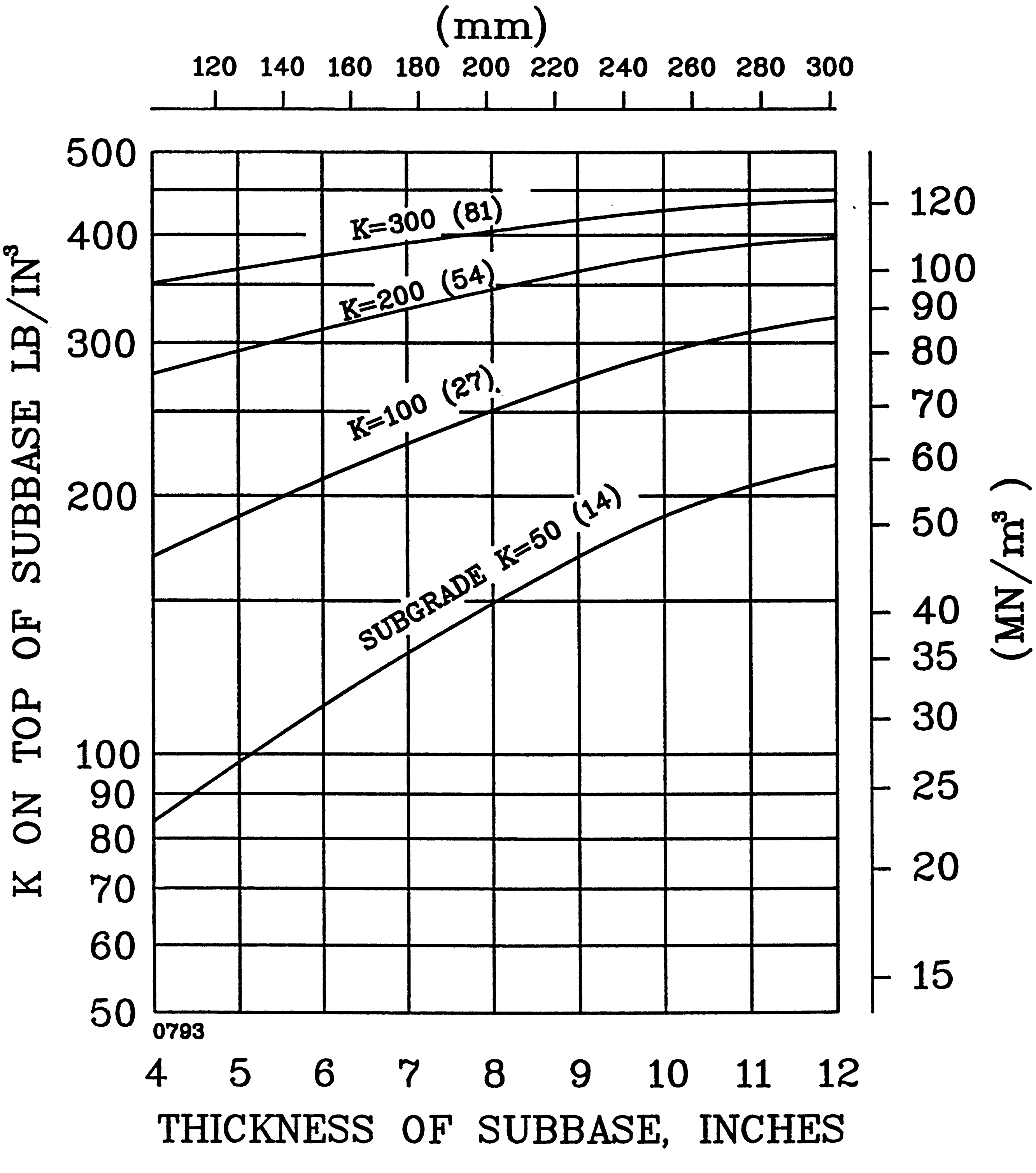


FIGURE 3-16 EFFECT OF STABILIZED SUBBASE ON SUBGRADE MODULUS

331. DETERMINATION OF CONCRETE SLAB THICKNESS. Design curves have been prepared for rigid pavements similar to those for flexible pavements; i.e., separate curves for a variety of landing gear types and aircraft. See Figures 3-17 through 3-29. These curves are based on a jointed edge loading assumption where the load is located either tangent or perpendicular to the joint. Use of the design curves requires four design input parameters: concrete flexural strength, subgrade modulus, gross weight of the design aircraft, and annual departure of the design aircraft. The rigid pavement design curves indicate the thickness of concrete only. Thicknesses of other components of the rigid pavement structure must be determined separately.

a. Concrete Flexural Strength. The required thickness of concrete pavement is related to the strength of the concrete used in the pavement. Concrete strength is assessed by the flexural strength, as the primary action of a concrete pavement slab is flexure. Concrete flexural strength should be determined by ASTM C 78 test method. The design flexural strength of the concrete should be based on the age and strength the concrete will be required to have when it is scheduled to be opened to traffic.

b. k Value. The k value is in effect, a spring constant for the material supporting the rigid pavement and is indicative of the bearing capacity of the supporting material.

c. Gross Weight of Design Aircraft. The gross weight of the design aircraft is shown on each design curve. The design curves are grouped in accordance with either main landing gear assembly type or as separate curves for individual aircraft. A wide range of gross weights is shown on all curves to assist in any interpolations which may be required. In all cases, the range of gross weights shown is adequate to cover weights of the aircraft represented.

d. Annual Departures of Design Aircraft. The fourth input parameter is annual departures of the design aircraft. The departures should be computed using the procedure explained in paragraph 305.

332. USE OF DESIGN CURVES.

a. Rigid Pavement Design Curves. The rigid pavement design curves are constructed such that the design inputs are entered in the same order as they are discussed in paragraph 331. Dashed "chase around lines" are shown on the curves to indicate the order of progression through the curves. Concrete flexural strength is the first input. The left ordinate of the design curve is entered with concrete flexural strength. A horizontal projection is made until it intersects with the appropriate foundation modulus line. A vertical projection is made from the intersection point to the appropriate gross weight of the design aircraft. A horizontal projection is made to the right ordinate showing annual departures. The pavement thickness is read from the appropriate annual departure line. The pavement thickness shown refers to the thickness of the concrete pavement only, exclusive of the subbase. This thickness is that shown as "T" in Figure 3-1, referred to as the critical thickness.

b. Optional Design Curves. When aircraft loadings are applied to a jointed edge, the angle of the landing gear relative to the jointed edge influences the magnitude of the stress in the slab. Single wheel and dual wheel landing gear assemblies produce the maximum stress when the gear is located parallel or perpendicular to the joint. Dual tandem assemblies often produce the maximum stress when positioned at an acute angle to the jointed edge. Figures 3-30 through 3-41, have been prepared for dual tandem gears located tangent to the jointed edge but rotated to the angle causing the maximum stress. These design curves can be used to design pavement in areas where aircraft are likely to cross the pavement joints at acute angles such as runway holding aprons, runway ends, runway-taxiway intersections, aprons, etc. Use of Figures 3-30 through 3-41 is optional and should only be applied in areas where aircraft are likely to cross pavement joints at an acute angle and at low speeds.

333. CRITICAL AND NONCRITICAL AREAS. The design curves, Figures 3-17 through 3-41, are used to determine the concrete slab thickness for the critical pavement areas shown as "T" in Figure 3-1. The 0.9T thickness for noncritical areas applies to the concrete slab thickness. For the variable thickness section of the thinned edge and transition section, the reduction applies to the concrete slab thickness. The change in thickness for the transitions should be accomplished over an entire slab length or width. In areas of variable slab thickness, the subbase thickness must be adjusted as necessary to provide surface drainage from the entire subgrade surface. For fractions of an inch of 0.5 or more, use the next higher whole number; for less than 0.5, use the next lower number.

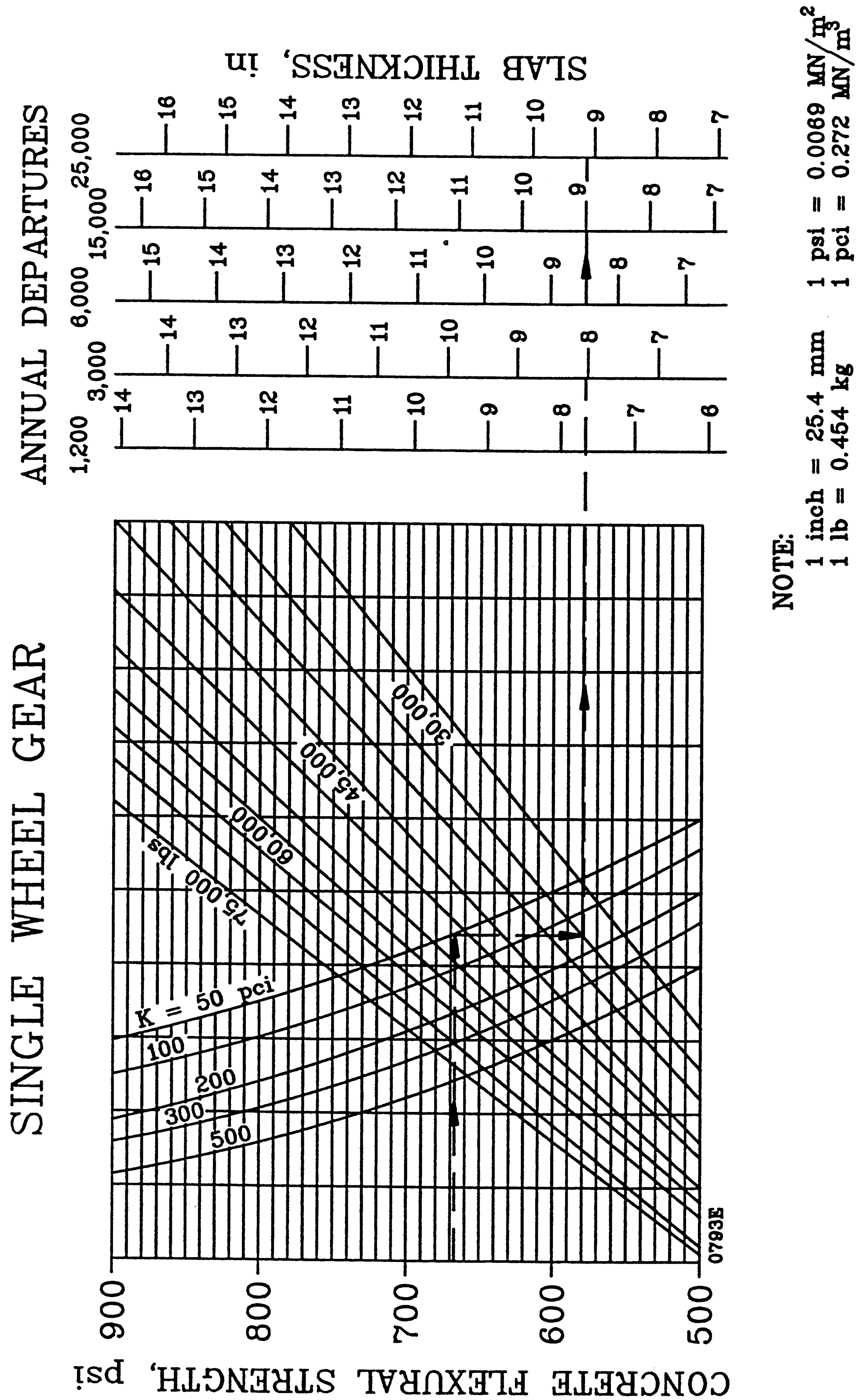
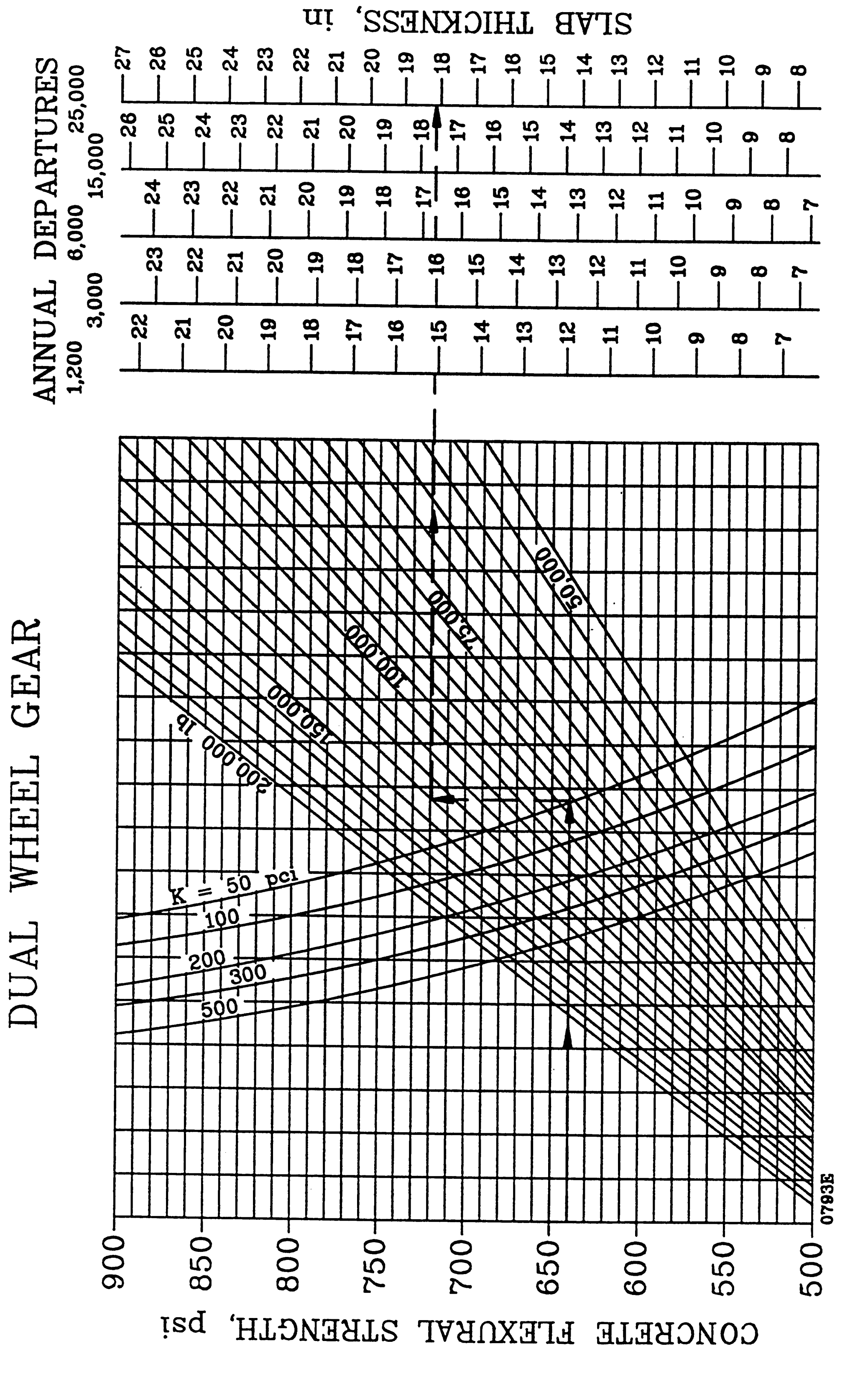
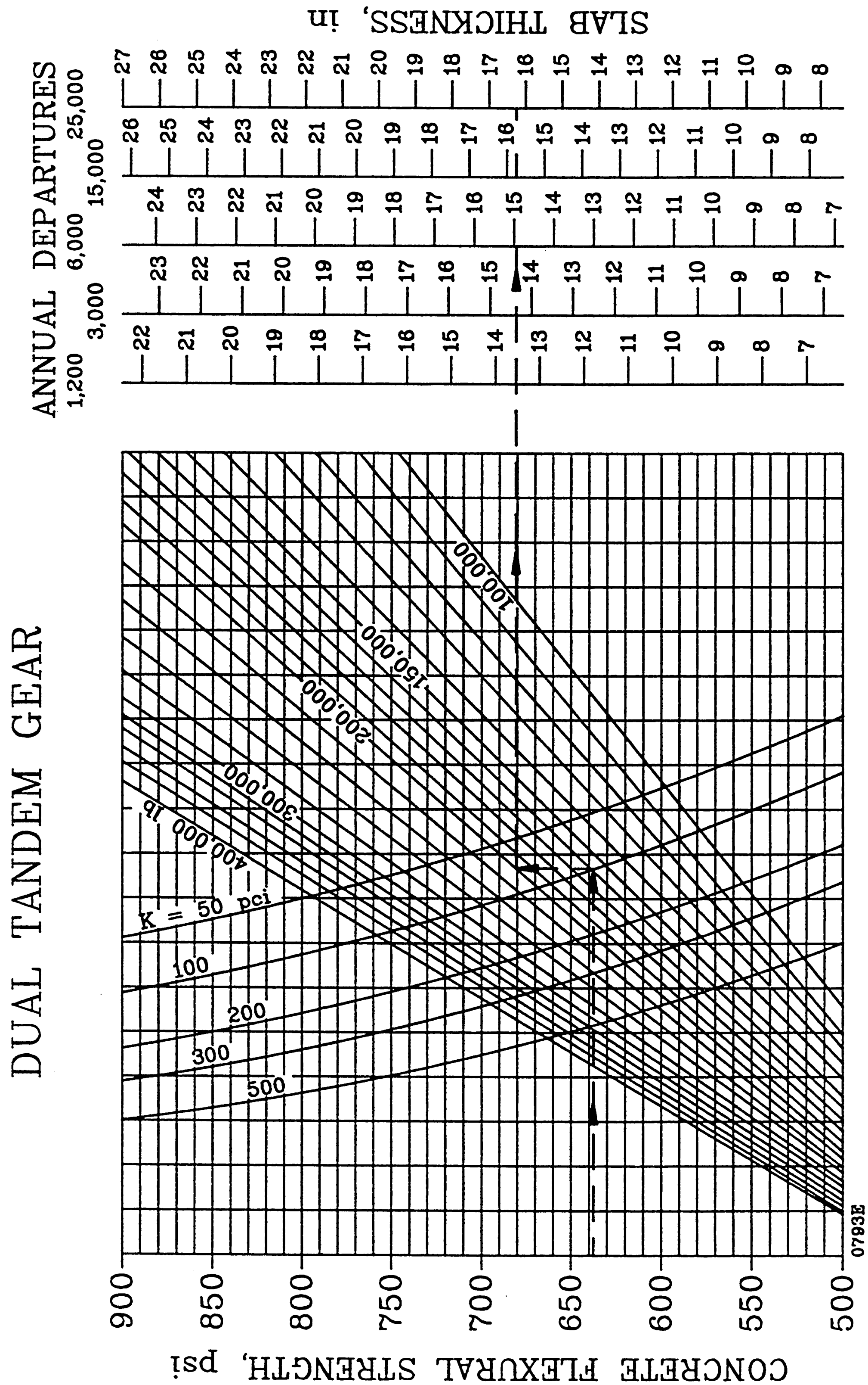


FIGURE 3-17. RIGID PAVEMENT DESIGN CURVES, SINGLE WHEEL GEAR



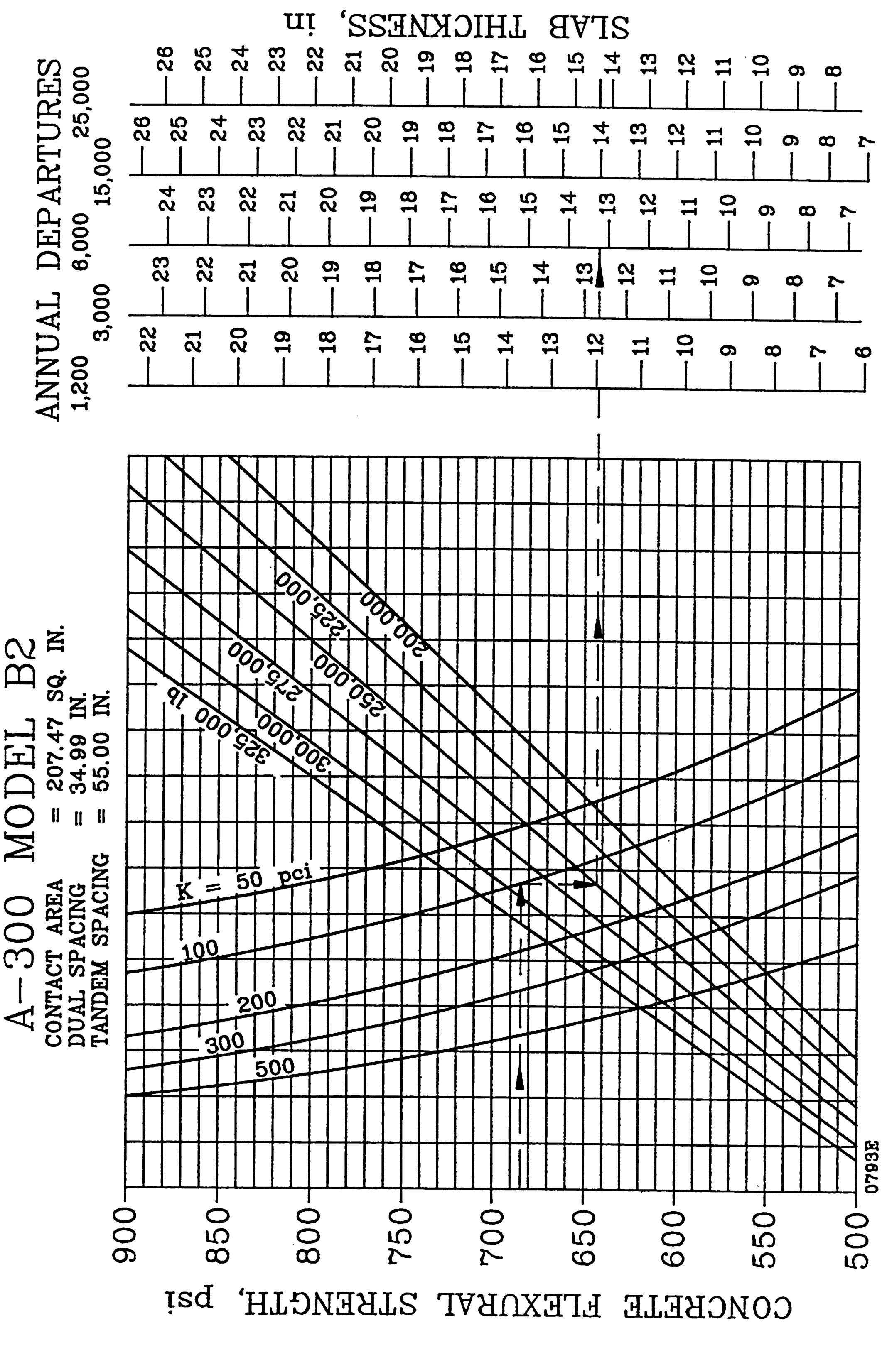
NOTE:
1 inch = 25.4 mm   1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg   1 pci = 0.272 MN/m^3

FIGURE 3-18. RIGID PAVEMENT DESIGN CURVES, DUAL WHEEL GEAR



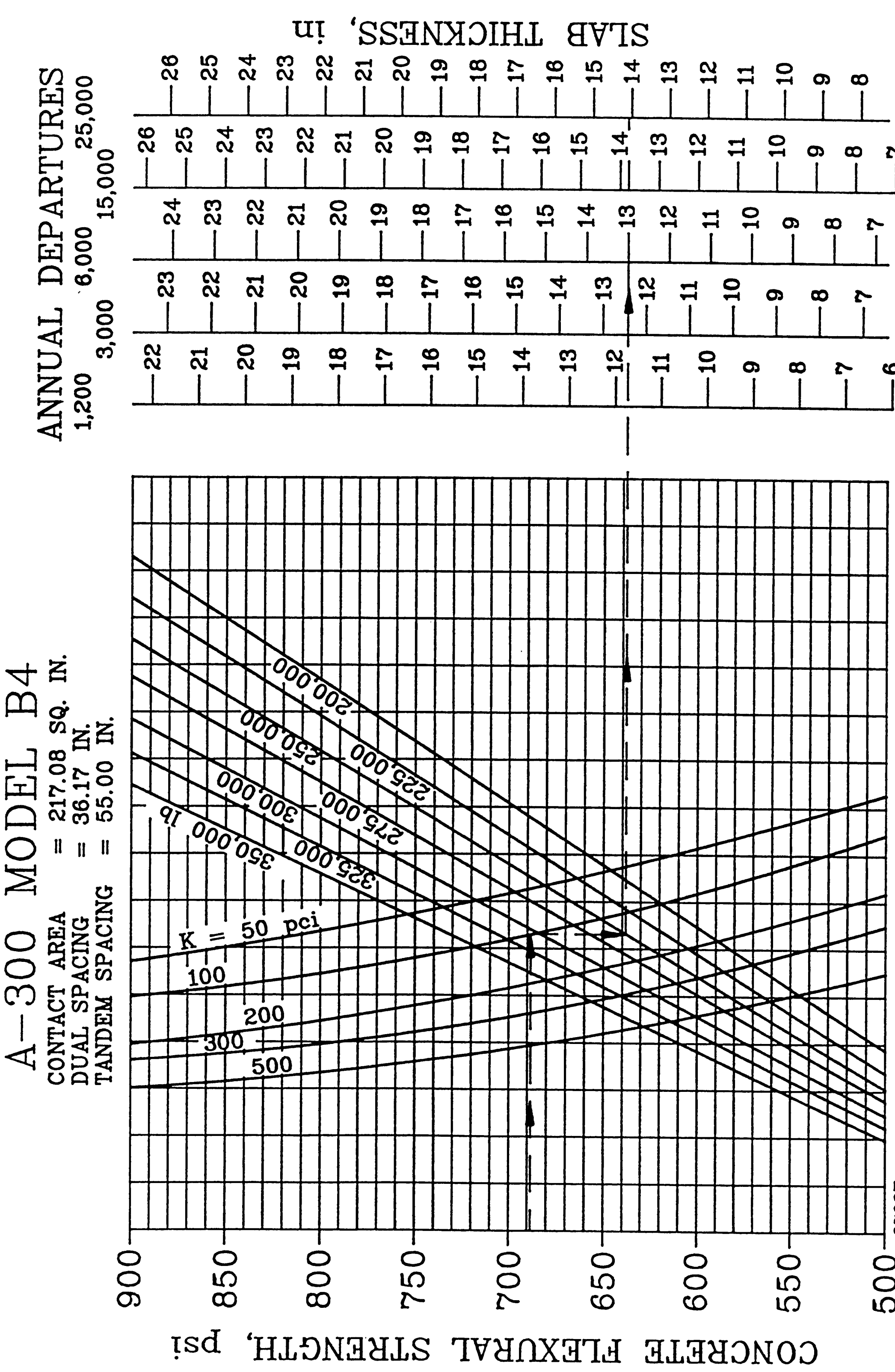
NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-19. RIGID PAVEMENT DESIGN CURVES, DUAL TANDEM GEAR



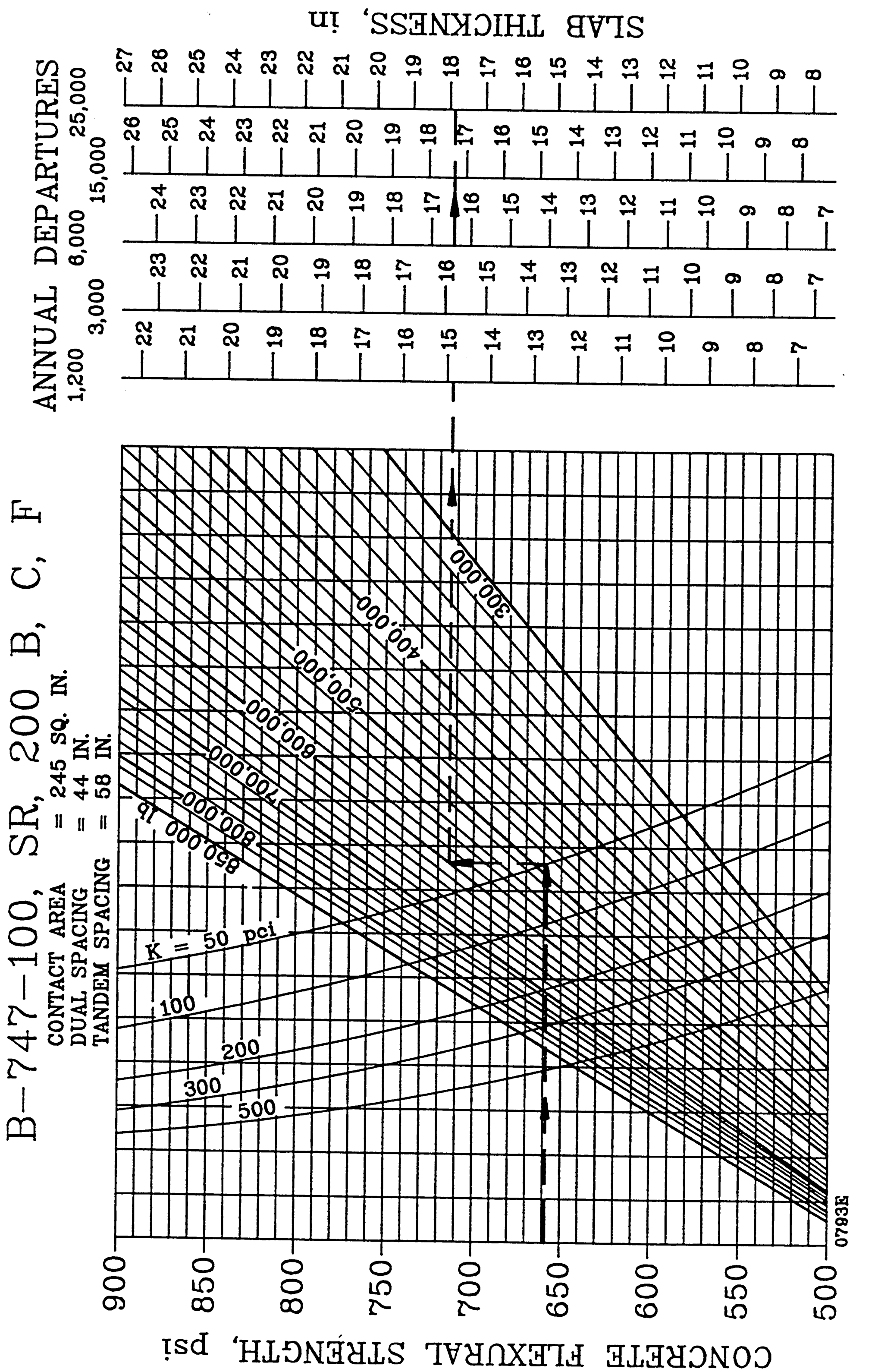
NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-20. RIGID PAVEMENT DESIGN CURVES, A-300 MODEL B2



NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-21. RIGID PAVEMENT DESIGN CURVES, A-300 MODEL B4



NOTE:
1 inch = 25.4 mm   1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg   1 pci = 0.272 MN/m^3

FIGURE 3-22. RIGID PAVEMENT DESIGN CURVES, B-747-100, SR, 200 B, C, F

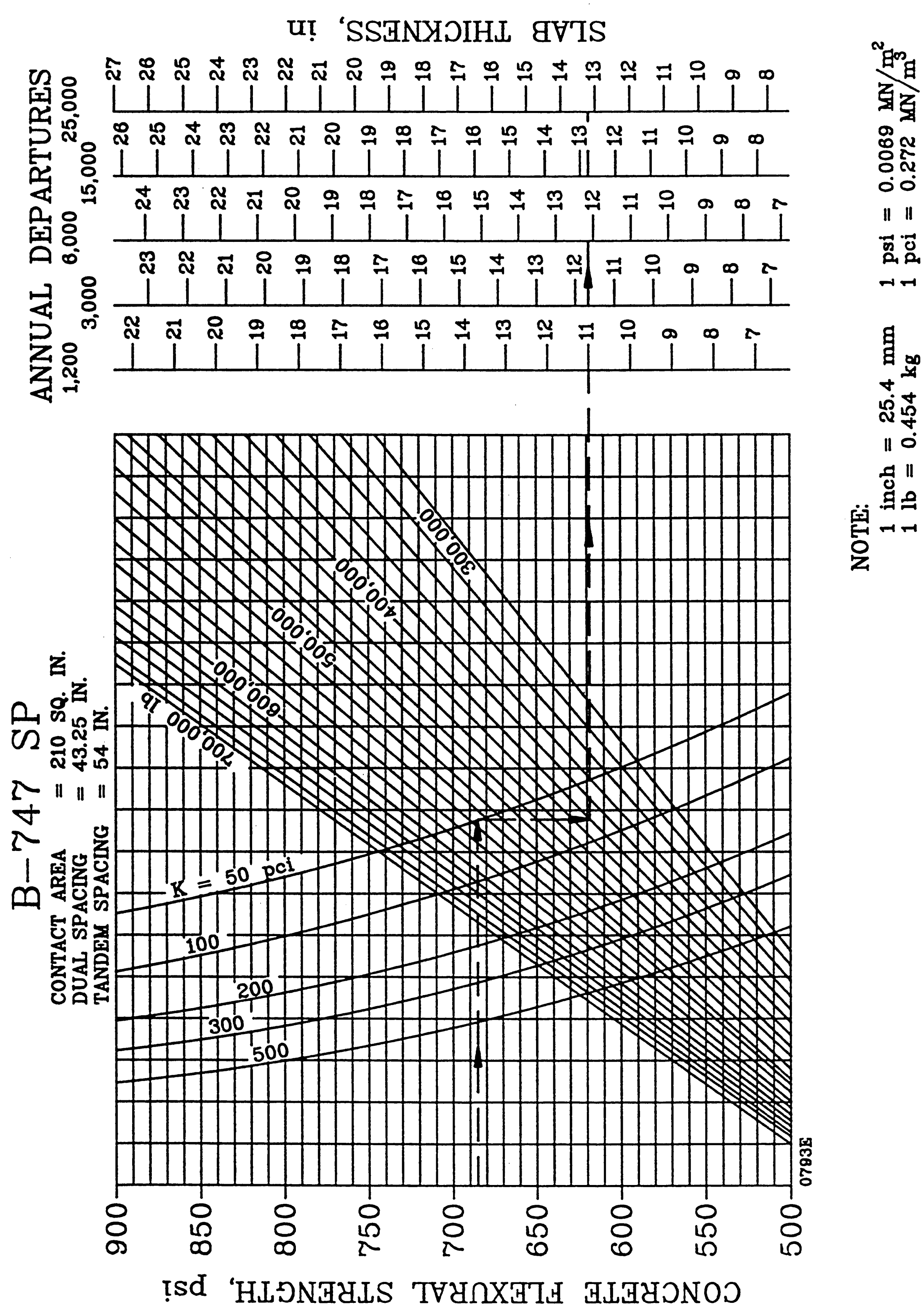
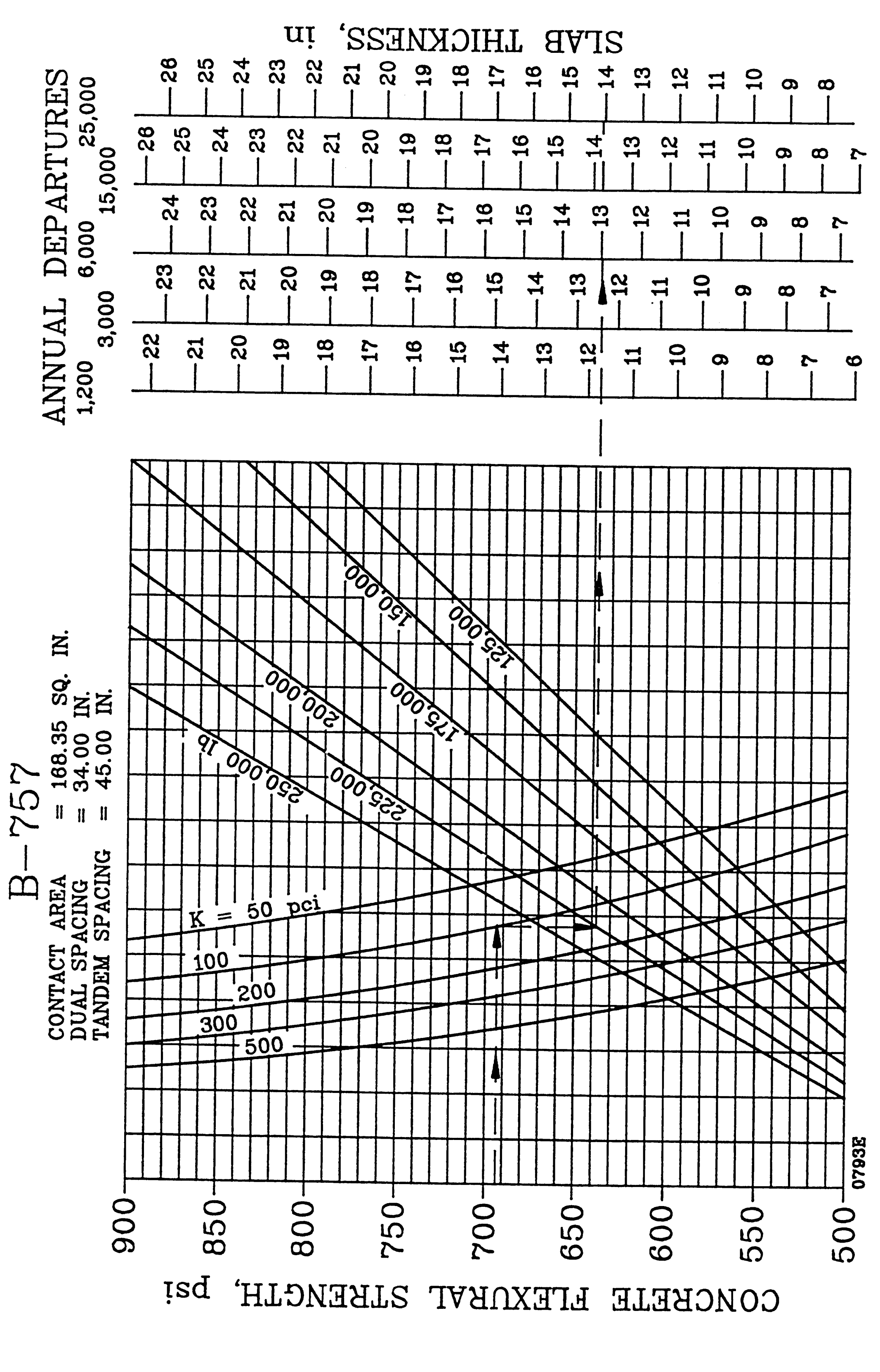
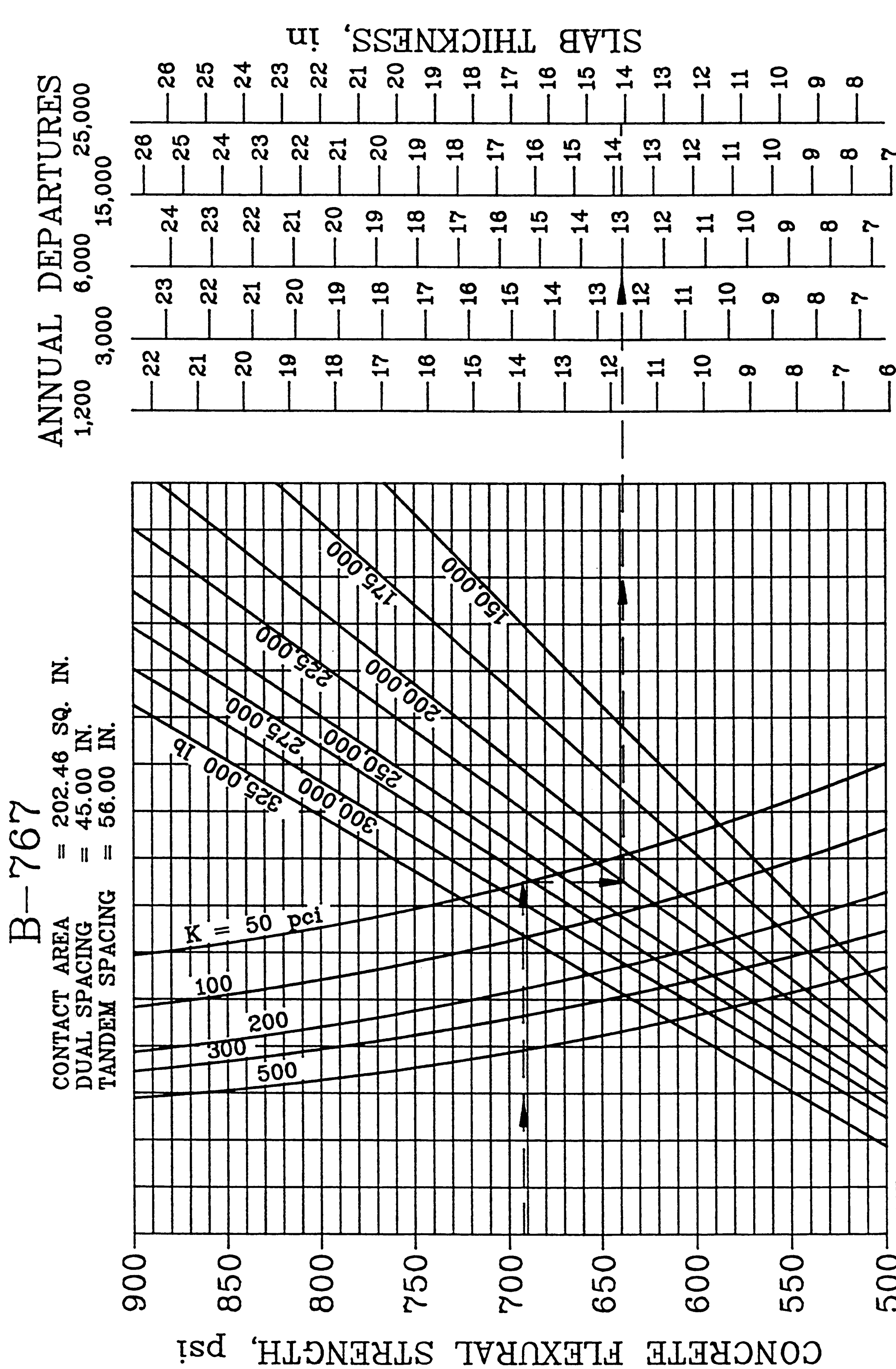


FIGURE 3-23. RIGID PAVEMENT DESIGN CURVES, B-747-SP



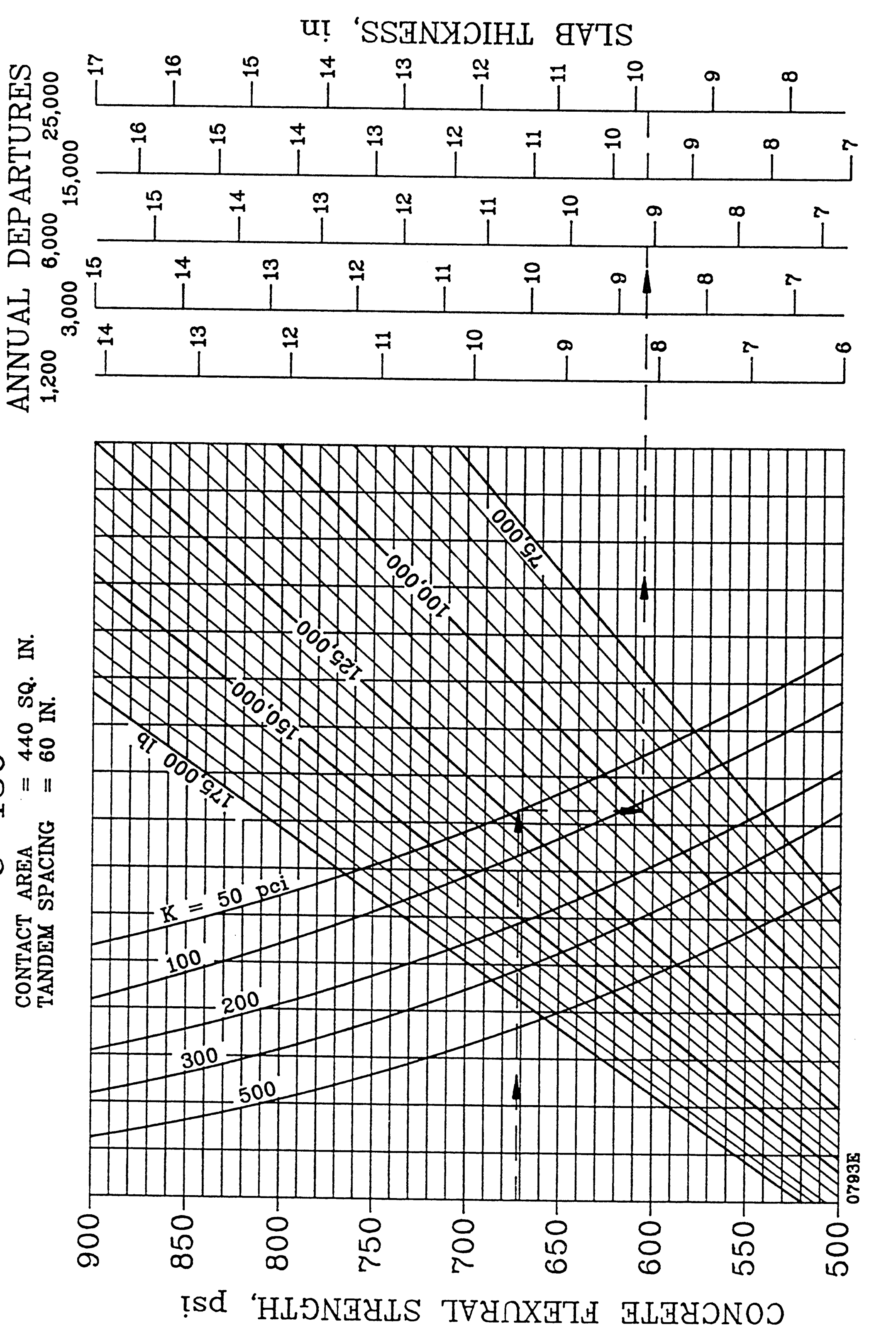
NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-24. RIGID PAVEMENT DESIGN CURVES, B-757



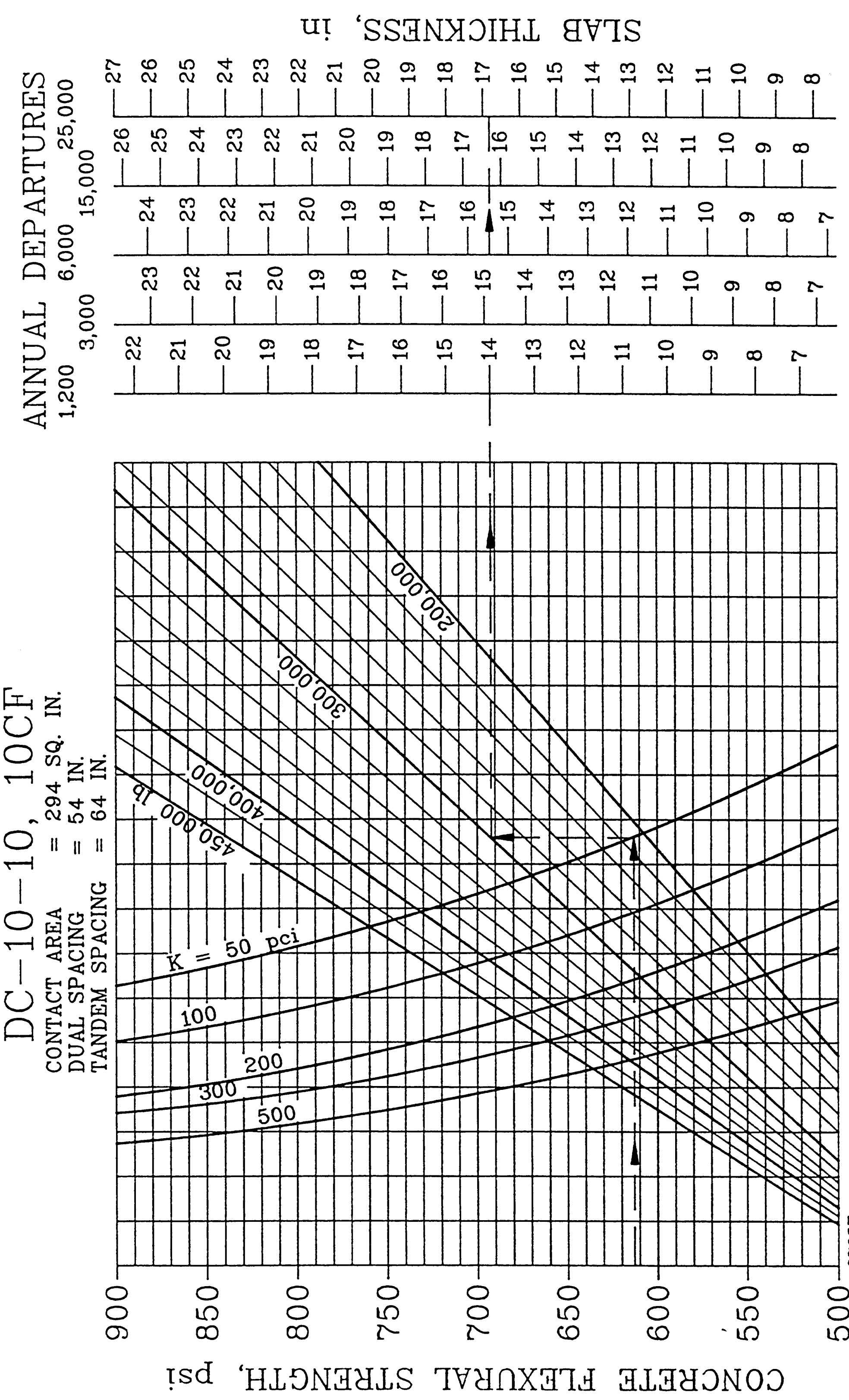
NOTE:
1 inch = 25.4 mm   1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg   1 pci = 0.272 MN/m^3

FIGURE 3-25. RIGID PAVEMENT DESIGN CURVES, B-767



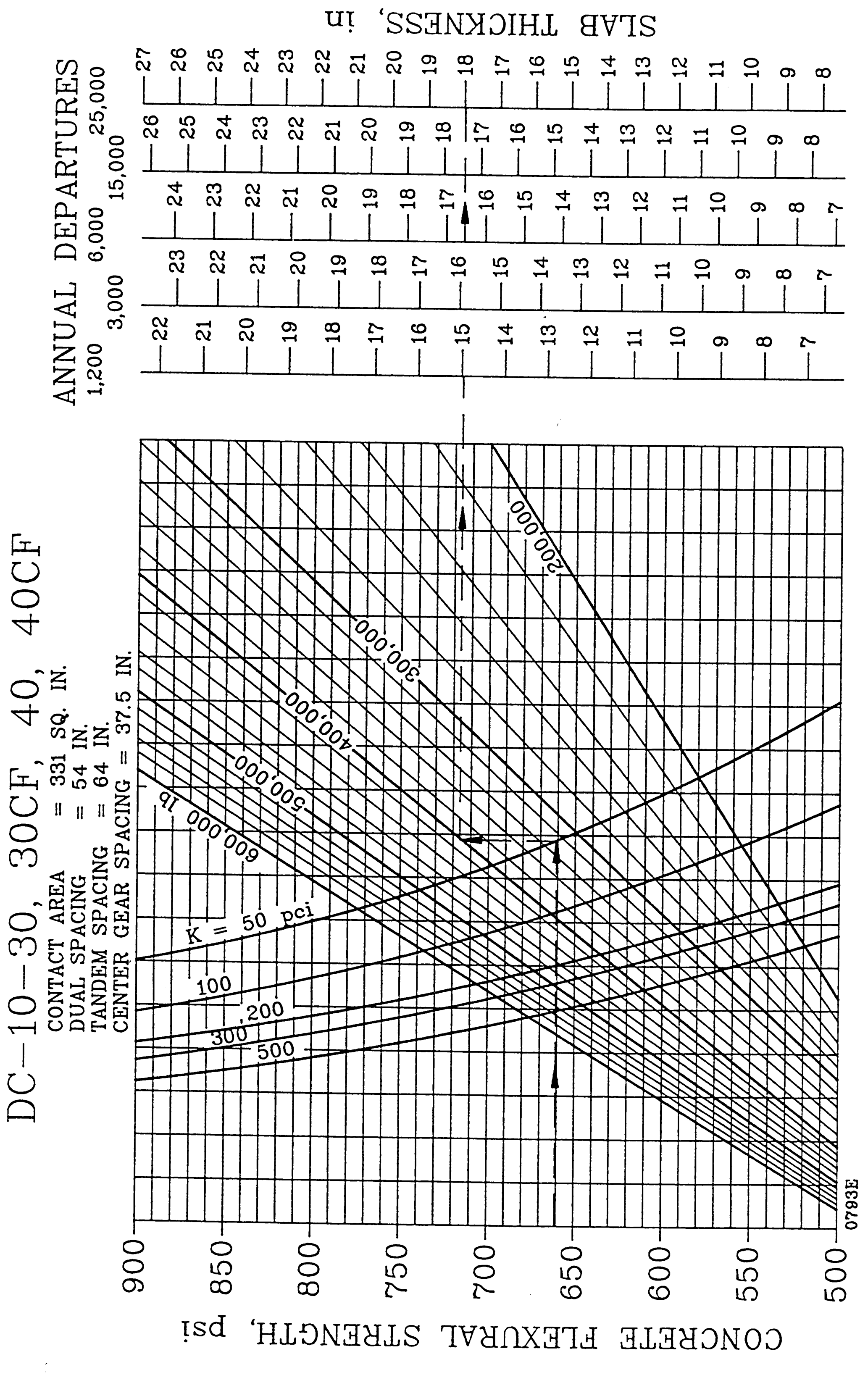
NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-26. RIGID PAVEMENT DESIGN CURVES, C-130



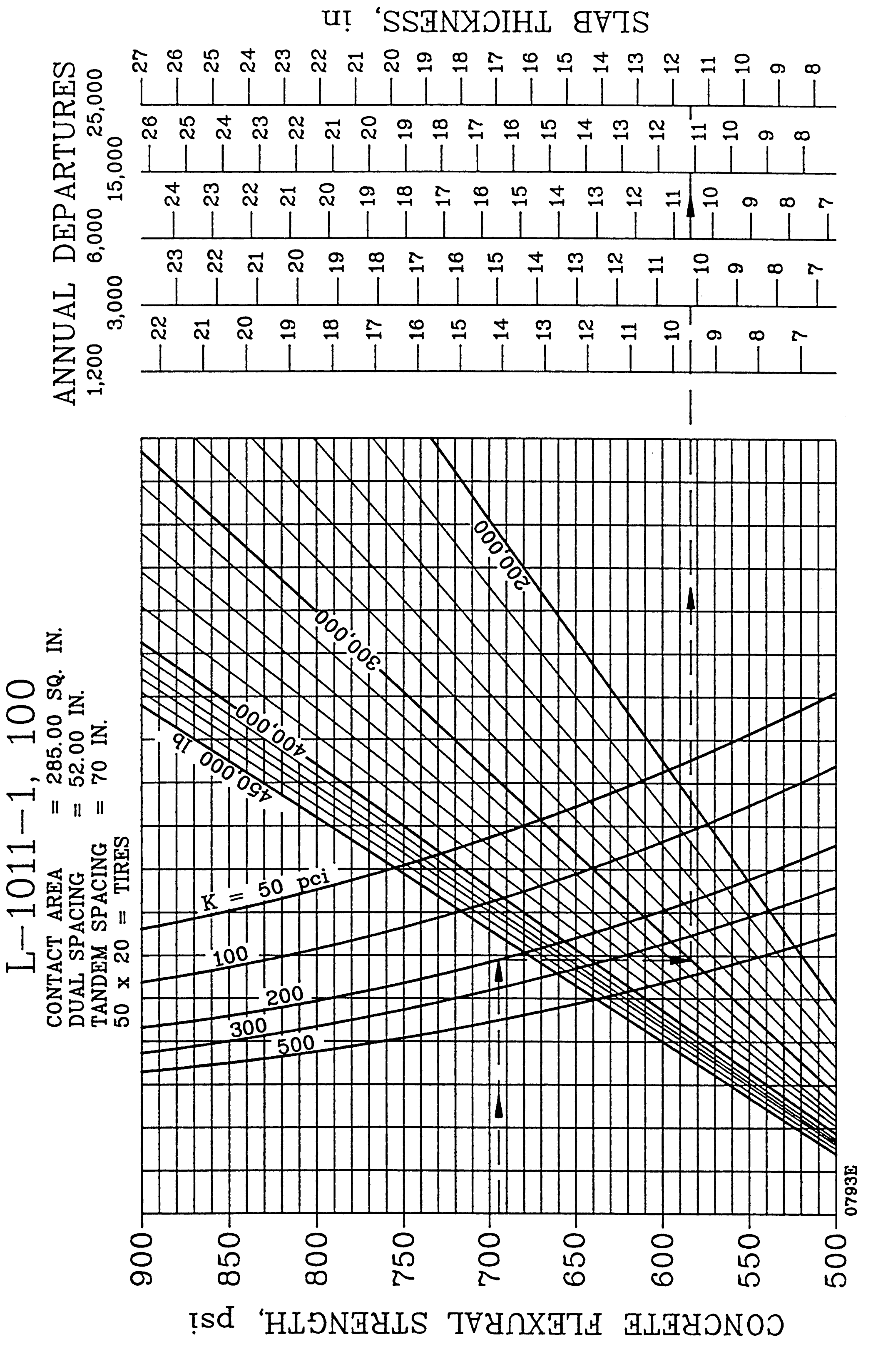
NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-27. RIGID PAVEMENT DESIGN CURVES, DC 10-10, 10CF



NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-28. RIGID PAVEMENT DESIGN CURVES, DC 10-30, 30CF, 40, 40CF



NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-29. RIGID PAVEMENT DESIGN CURVES, L-1011-1, 100

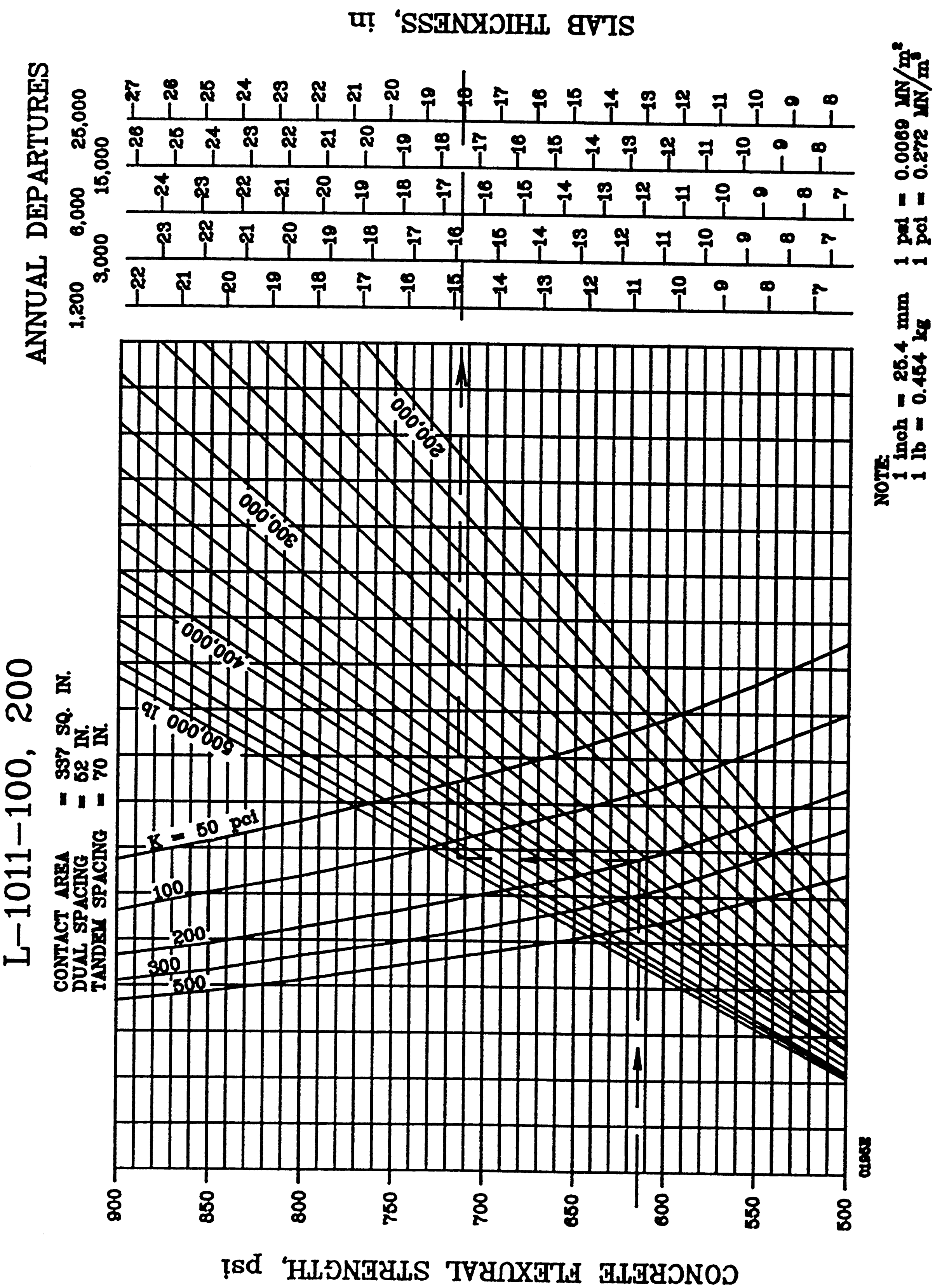
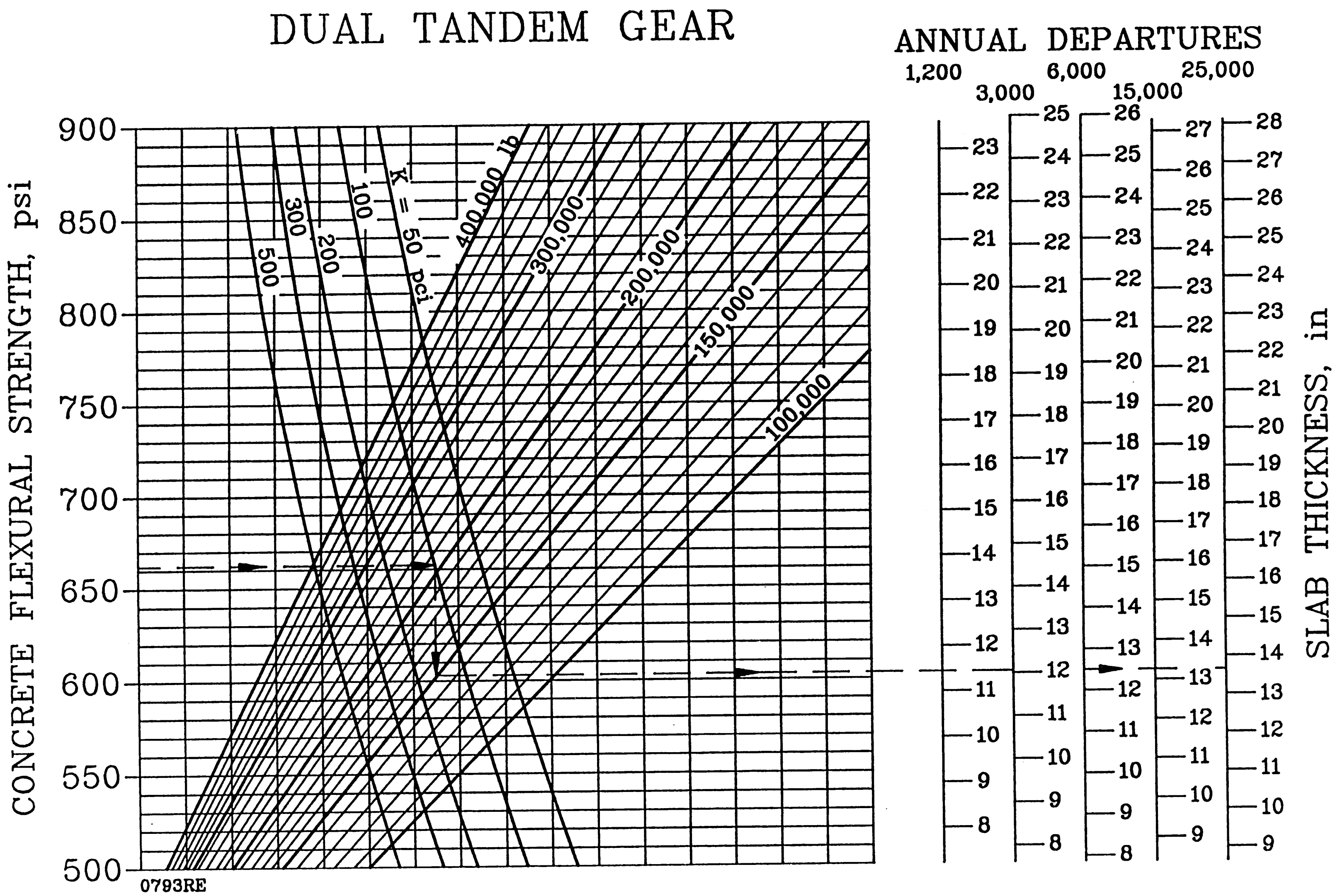
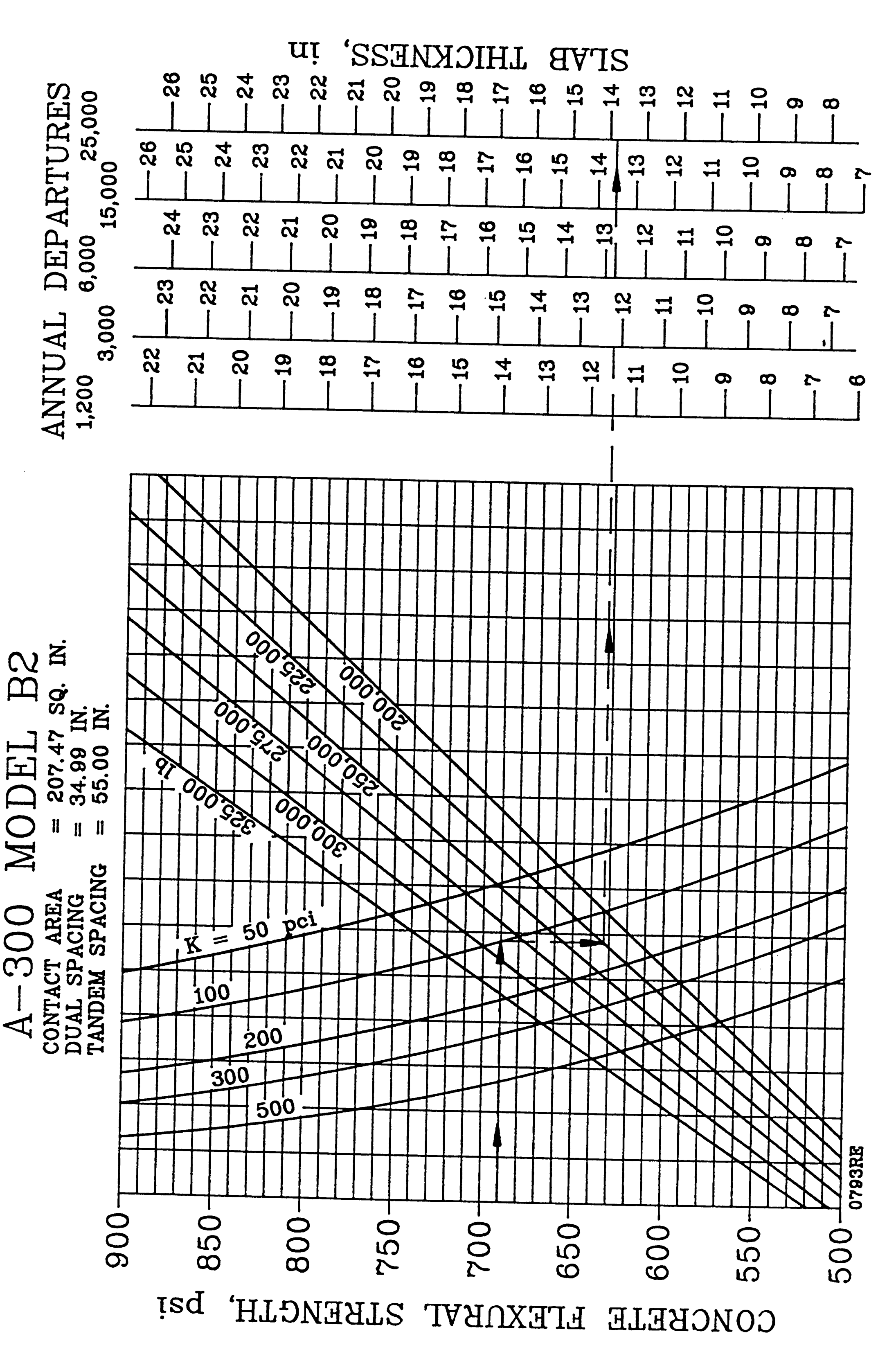


FIGURE 3-30. RIGID PAVEMENT DESIGN CURVES, L-1011-100, 200



NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-31. OPTIONAL RIGID PAVEMENT DESIGN CURVES, DUAL TANDEM GEAR



NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-32. OPTIONAL RIGID PAVEMENT DESIGN CURVES, A-300 MODEL B2

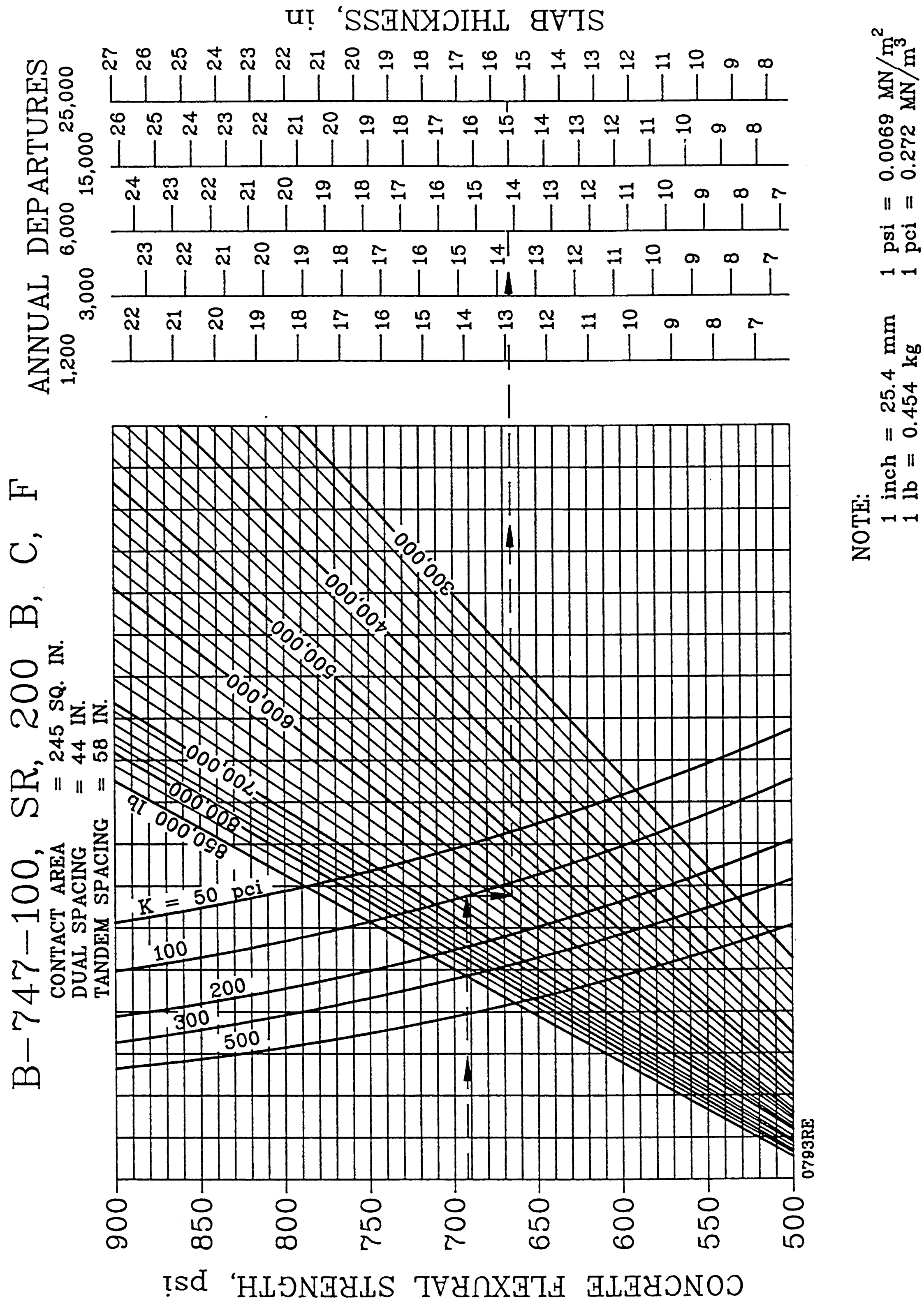
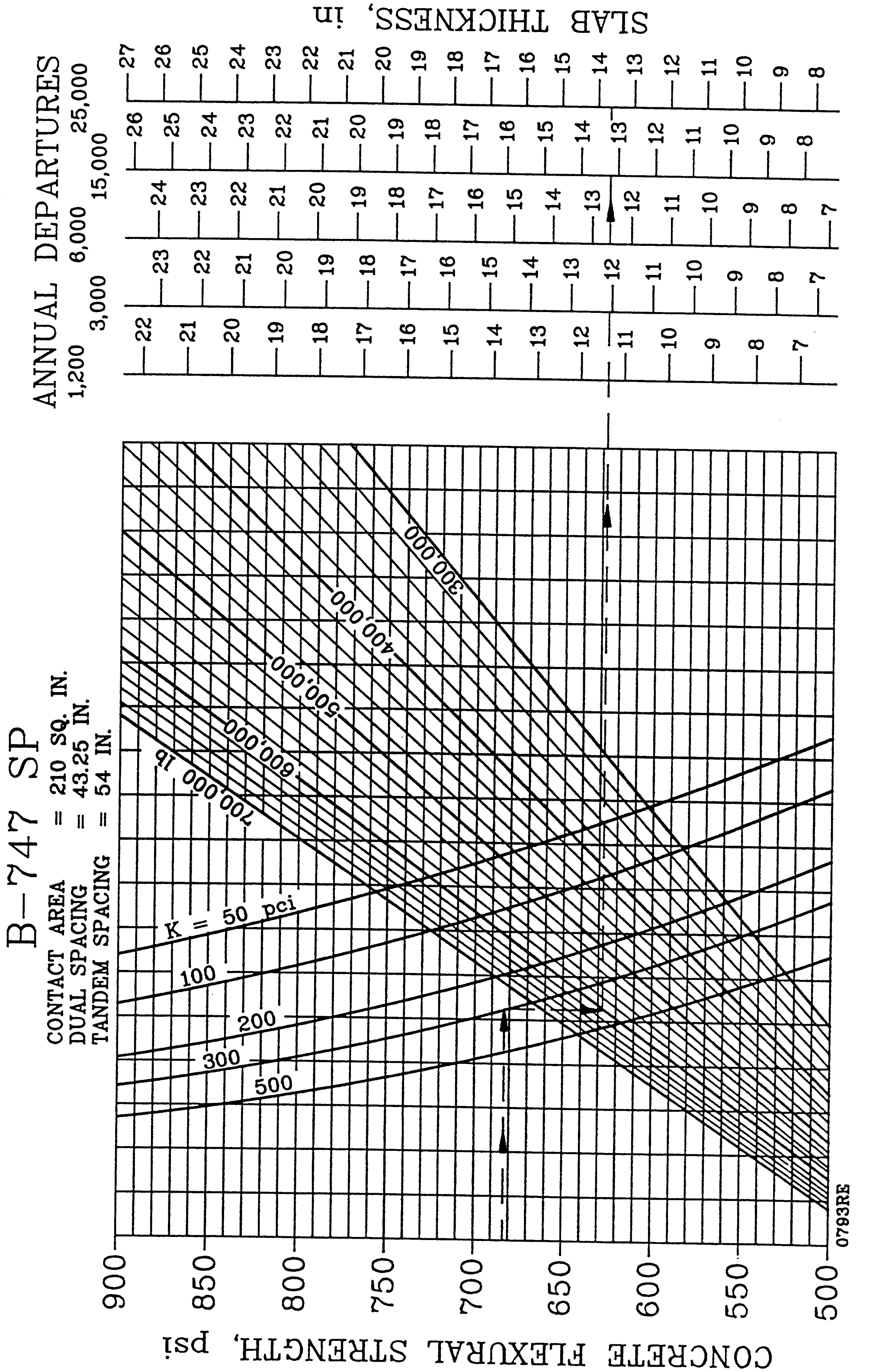
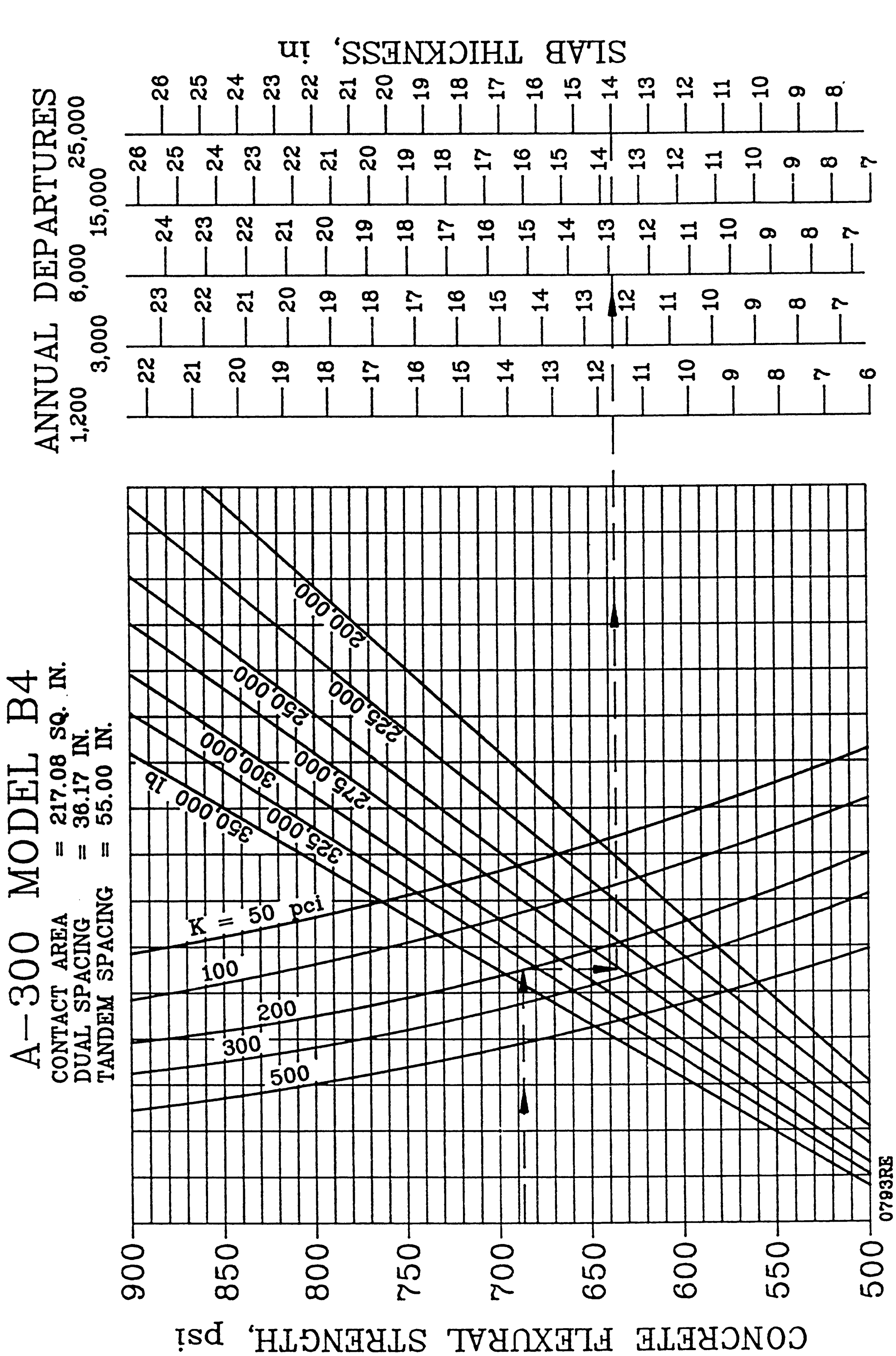


FIGURE 3-33. OPTIONAL RIGID PAVEMENT DESIGN CURVES, B-747-100, SR, 200 B, C, F



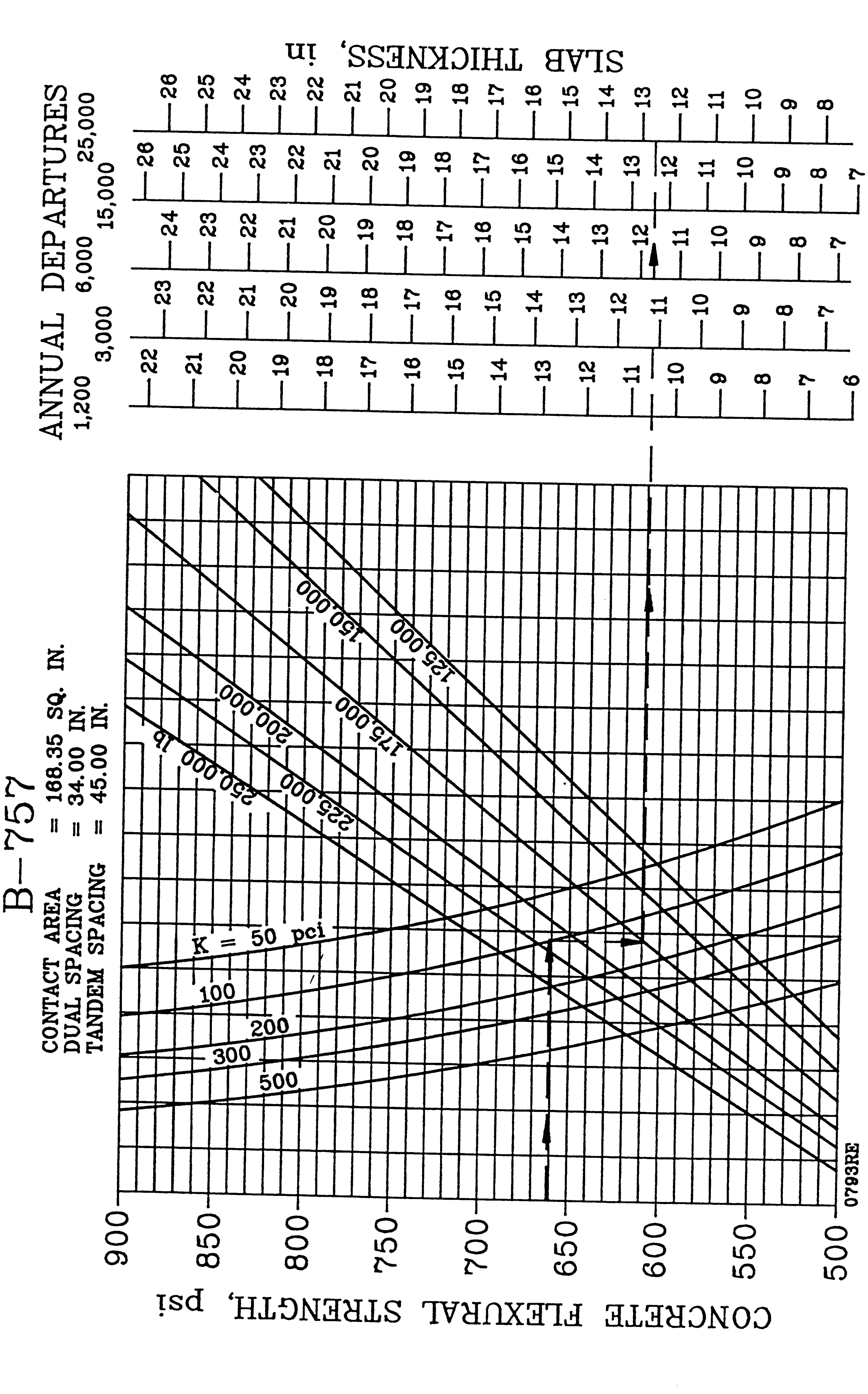
NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-34. OPTIONAL RIGID PAVEMENT DESIGN CURVES, B-747-SP



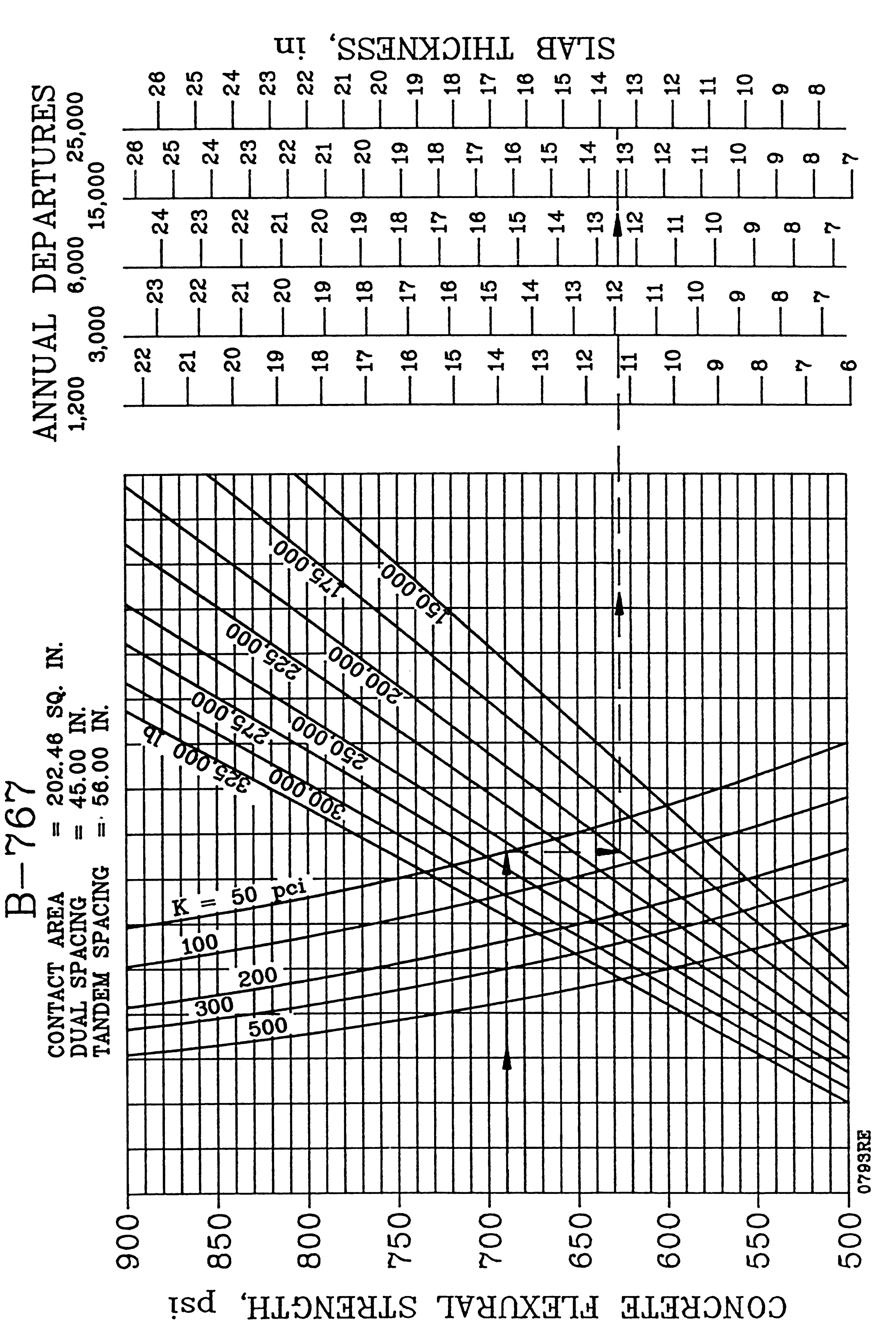
NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-35. OPTIONAL RIGID PAVEMENT DESIGN CURVES, A-300 Model B4



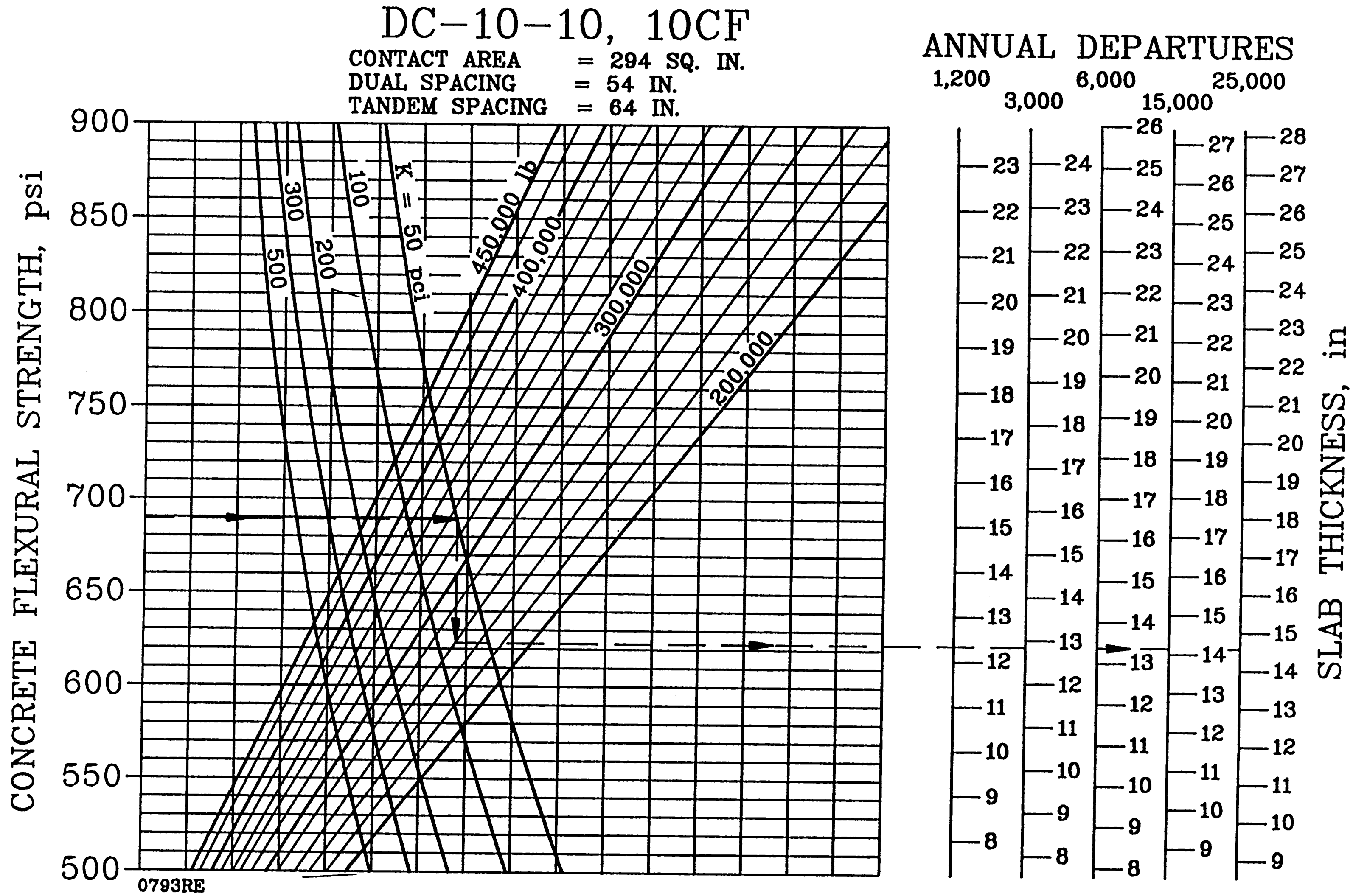
NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-36. OPTIONAL RIGID PAVEMENT DESIGN CURVES, B-757



NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-37. OPTIONAL RIGID PAVEMENT DESIGN CURVES, B-767



NOTE:
1 inch = 25.4 mm   1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg   1 pci = 0.272 MN/m^3

FIGURE 3-38. OPTIONAL RIGID PAVEMENT DESIGN CURVES, DC10-10, 10CF

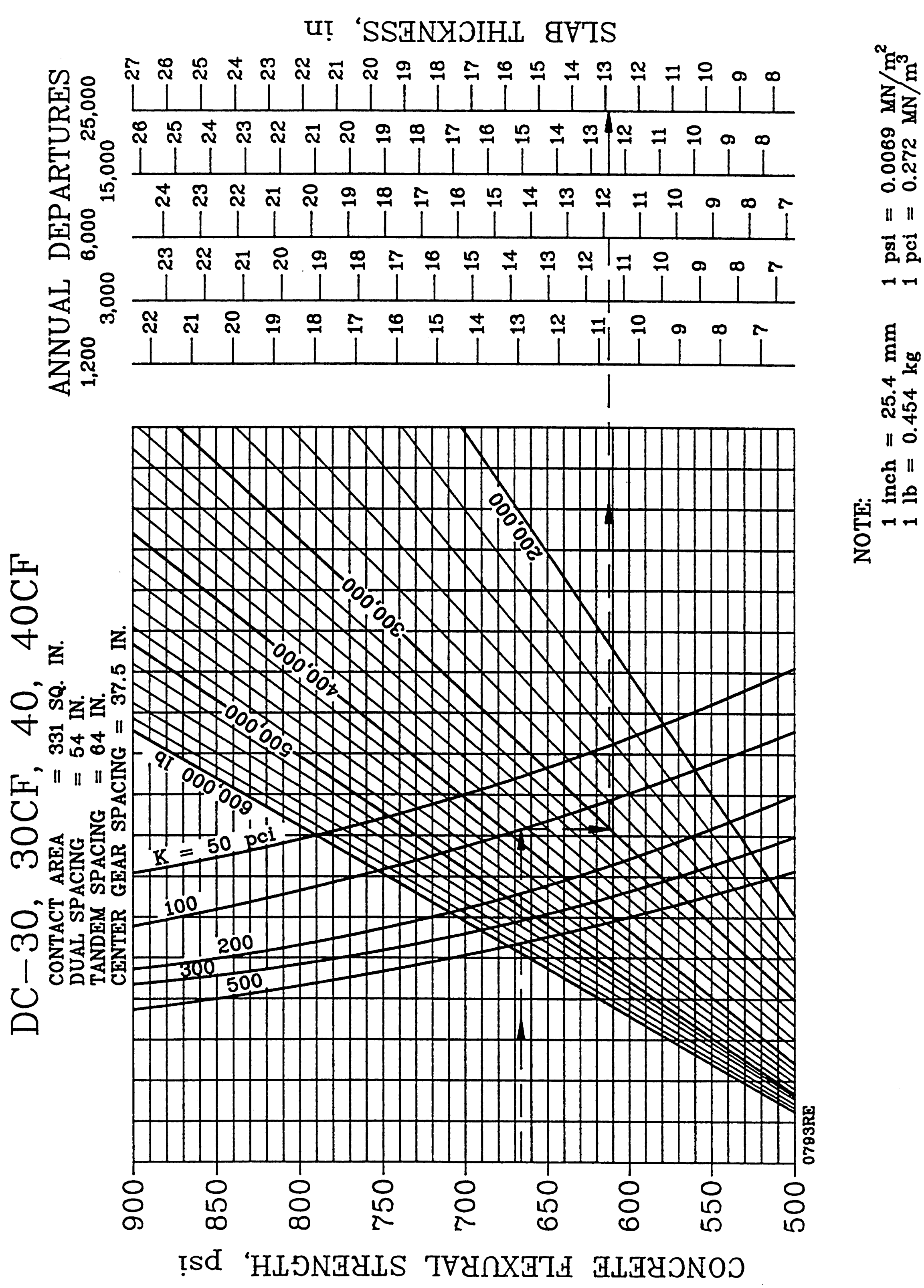
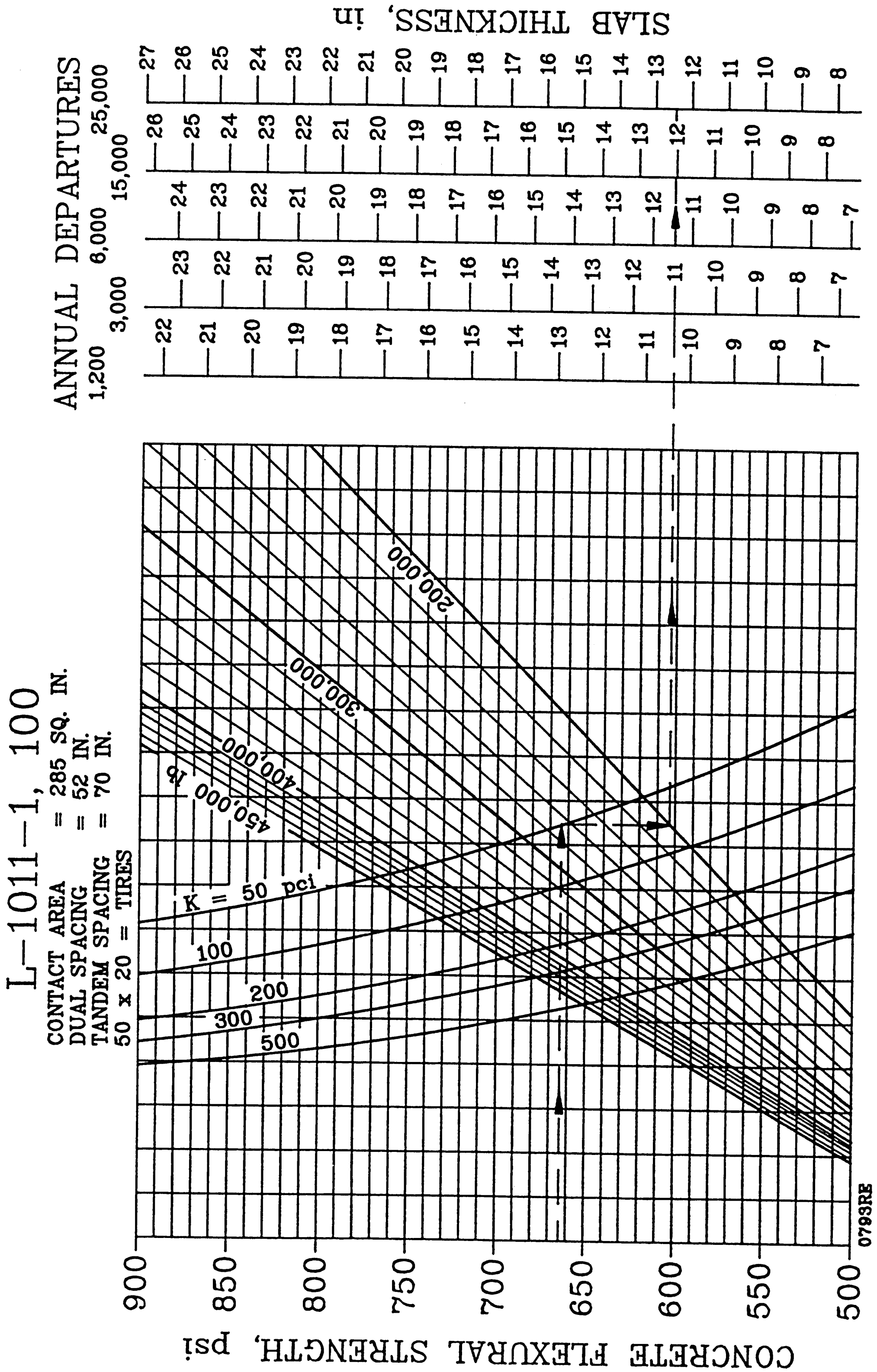


FIGURE 3-39. OPTIONAL RIGID PAVEMENT DESIGN CURVES, DC-10-30, 30CF, 40, 40CF



NOTE:
1 inch = 25.4 mm 1 psi = 0.0069 MN/m^2
1 lb = 0.454 kg 1 pci = 0.272 MN/m^3

FIGURE 3-40. OPTIONAL RIGID PAVEMENT DESIGN CURVES, L-1011-1, 100

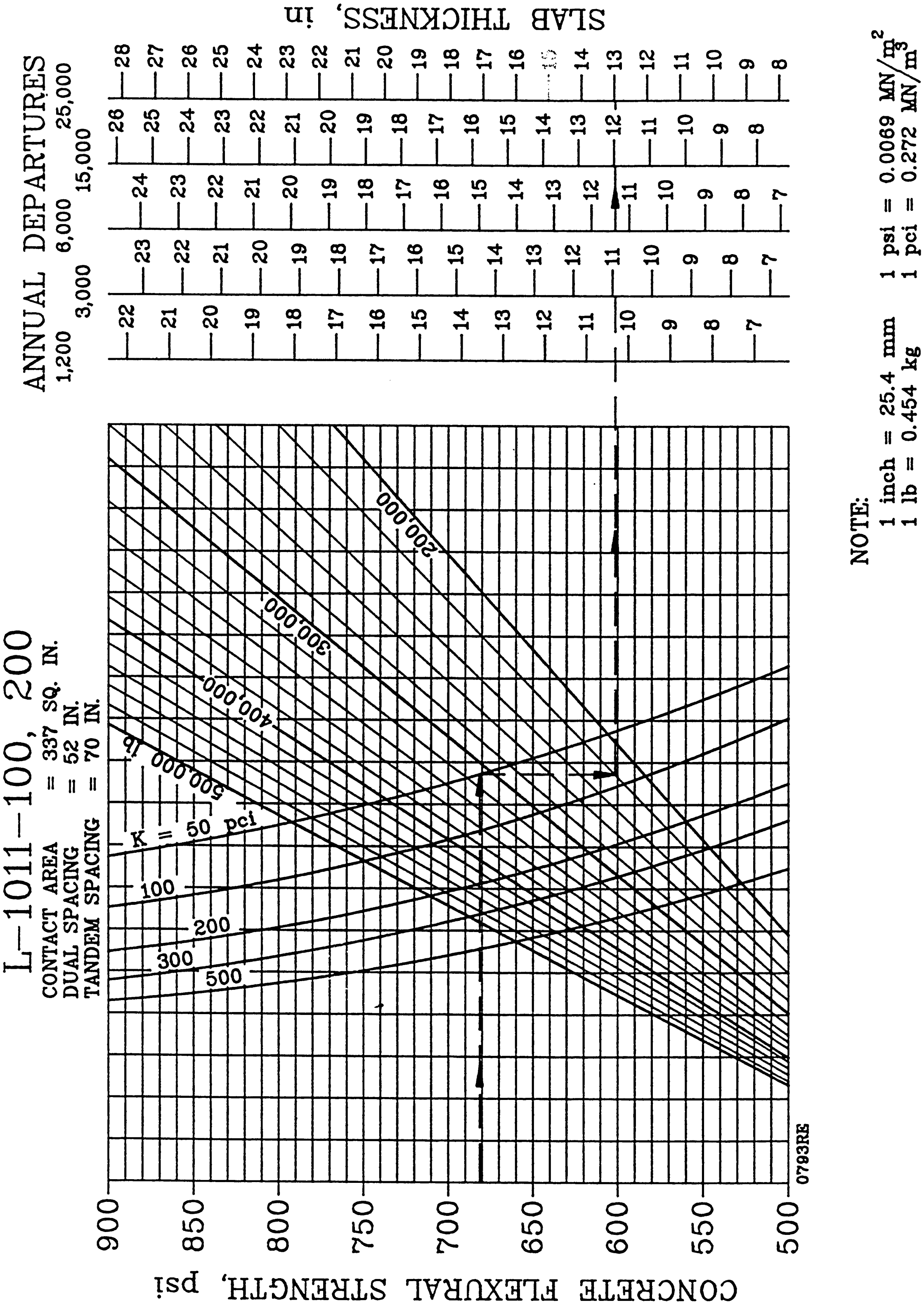


FIGURE. 3-41. OPTIONAL PAVEMENT DESIGN CURVES, L-1011-100, 200

334. DESIGN EXAMPLE. As an example of the use of the design curves, assume that a rigid pavement is to be designed for dual tandem aircraft having a gross weight of 350,000 pounds (160 000 kg) and for 6,000 annual equivalent departures of the design aircraft. The equivalent annual departures of 6,000 include 1,200 annual departures of B-747 aircraft weighing 780,000 pounds (350 000 kg) gross weight. The subgrade modulus of 100 PCI (25 MN/m^3) with poor drainage and frost penetration is 18 inches (460 mm). The feature to be designed is a primary runway and requires 100 percent frost protection. The subgrade soil is CL. Concrete mix designs indicate a flexural strength of 650 PSI (4.5 MN/m^2) can be readily produced with locally available aggregates. The gross weight of the design aircraft dictates the use of a stabilized subbase. Several thicknesses of stabilized subbases should be tried to determine the most economical section. Assume a stabilized subbase of P-304 will be used. Try a subbase thickness of 6 inches (150 mm). Using Figure 3-16, a 6-inch (150 mm) thickness of P-304 would likely increase the foundation modulus from 100 PCI (25 MN/m^3) to 210 PCI (57 MN/m^3). Using Figure 3-19, dual tandem design curve, with the assumed design data, yields a concrete pavement thickness of 16.6 inches (422 mm). This thickness would be rounded off to 17 inches (430 mm). Since the frost penetration is only 18 inches (460 mm) and the combined thickness of concrete pavement and stabilized subbase is 23 inches (585 mm), no further frost protection is needed. Even though the wide body aircraft did not control the thickness of the slab, the wide bodies would have to be considered in the establishment of jointing requirements and design of drainage structures. Other stabilized subbase thicknesses should be tried to determine the most economical section.

335. FROST EFFECTS. As with flexible pavements, frost protection should be provided for rigid pavements in areas where conditions conducive to detrimental frost action exist. Frost protection considerations for rigid pavements are similar to those for flexible pavements. The determination of the depth of frost protection required is given in paragraph 308.b. Local experience may be used to refine the calculations.

a. Example. Assume the above design example is for a primary runway and requires complete frost protection. The subgrade soil is CL, weighing 115 lbs/cu ft (184 kg/cu m). The design freezing index is 500 degree days. Referring to Figure 2-6 shows the depth of frost penetration to be 34 inches (865 mm). The structural considerations yield a 23 inch (585 mm) thickness of non-frost susceptible material. Since the frost penetration is only 18 inches (460 mm) and the combined thickness of concrete pavement and stabilized subbase is 23 inches (585 mm), no further frost protection is needed. Even though the wide body aircraft did not control the thickness of the slab, the wide bodies would have to be considered in the establishment of jointing requirements and design of drainage structures. Other stabilized subbase thicknesses should be tried to determine the most economical section.

(1) Complete Frost Protection. The complete frost protection method applies only to FG-3 and FG-4 soils which are extremely variable in horizontal extent. These soil deposits are characterized by very large, frequent, and abrupt changes in frost heave potential. The variability is such that the use of transition sections is not practical.

(2) Limited Subgrade Frost Penetration. This design method should be used for FG-4 soils except where the conditions require complete protection, see (1) above. The method also applies to soils in frost groups FG-1, FG-2, and FG-3 when the functional requirements of the pavement permit a minor amount of frost heave. Consideration should be given to using transition sections where horizontal variability of frost heave potential permits.

(3) Reduced Subgrade Strength. The reduced subgrade strength method is recommended for FG-1, FG-2, and FG-3 subgrades which are uniform in horizontal extent or where the functional requirements of the pavement will permit some degree of frost heave. the method may also be used for variable FG-1 through FG-3 subgrades for less sensitive pavements which are subject to slow speed traffic and heave can be tolerated.

336. HIGH TRAFFIC VOLUMES. There are a number of airports which experience traffic intensities in excess of those indicated on the design curves. Pavement maintenance is difficult and costly at high activity airports due to traffic intensity and the potential for aircraft delays. Performance of airport pavements under high traffic intensities has been reported in FAA-PM-84/14 (see Appendix 4). Rigid pavement designed to serve in situations where traffic intensity is high should reflect the following considerations.

a. Foundation. The foundation for the pavement provides the ultimate support to the structure. Every effort should be made to provide a stable foundation as problems arising later from an inadequate foundation cannot be practicably corrected after the pavement is constructed. The use of stabilized subbase will aid greatly in providing a uniform, stable foundation.

b. Thickness. Pavements subjected to traffic intensities greater than the 25,000 annual departure level shown on the design curves will require more thickness to accommodate the traffic volume. Additional thickness can be provided by increasing the pavement thickness in accordance with Table 3-5.

c. Panel Size. Slab panels should be constructed to minimize joint movement. Panel sizes given in paragraph 336 should be selected conservatively. Small joint movement tends to provide for better load transfer across joints and reduces the elongation the joint sealant materials must accommodate when the slabs expand and contract. High quality joint sealants should be specified to provide the best possible performance.

337. JOINTING OF CONCRETE PAVEMENTS. Variations in temperature and moisture content can cause volume changes and slab warping resulting in significant stresses. In order to reduce the detrimental effects of these stresses and to minimize random cracking, it is necessary to divide the pavement into a series of slabs of predetermined dimensions by means of joints. These slabs should be as nearly square as possible when no reinforcement is used.

a. Joint Categories. Pavement joints are categorized according to the function which the joint is intended to perform. The categories are expansion, contraction, and construction joints. All joints, regardless of type, should be finished in a manner which permits the joint to be sealed. Pavement joint details are shown in Figure 3-42 and are summarized in Table 3-10. These various joints are described as follows:

(1) Expansion Joints. The function of an expansion joint is to isolate intersecting pavements and to isolate structures from the pavement. There are two types of expansion joints.

(i) Type A. Type A is used when load transfer across the joint is required. This joint contains a 3/4-inch (19 mm) nonextruding compressible material and is provided with dowel bars for load transfer.

(ii) Type B. Type B is used when conditions preclude the use of load transfer devices which span across the joint, such as where the pavement abuts a structure or where horizontal differences in movement of the pavements may occur. These joints are formed by increasing the thickness of the pavement along the edge of slab. No dowel bars are provided.

(2) Contraction Joints. The function of contraction joints is to provide controlled cracking of the pavement when the pavement contracts due to decrease in moisture content, or a temperature drop. Contraction joints also decrease stresses caused by slab warping. Details for contraction joints are shown as Types F, G, and H, in Figure 3-42.

(3) Construction Joints. Construction joints are required when two abutting slabs are placed at different times such as at the end of a day's placement, or between paving lanes. Details for construction joints are shown as Types C, D, and E in Figure 3-42.

b. Joint Spacing.

(1) Without Stabilized Subbase. A rule-of-thumb for joint spacing given by the Portland Cement Association is applicable for rigid pavements without stabilized subbase: "As a rough guide, the joint spacing (in feet) should not greatly exceed twice the slab thickness (in inches)." Table 3-11 shows the recommended maximum joint spacings. Shorter spacings may be more convenient in some instances. The ratio of slab length to slab width should not exceed 1.25 in unreinforced pavements.

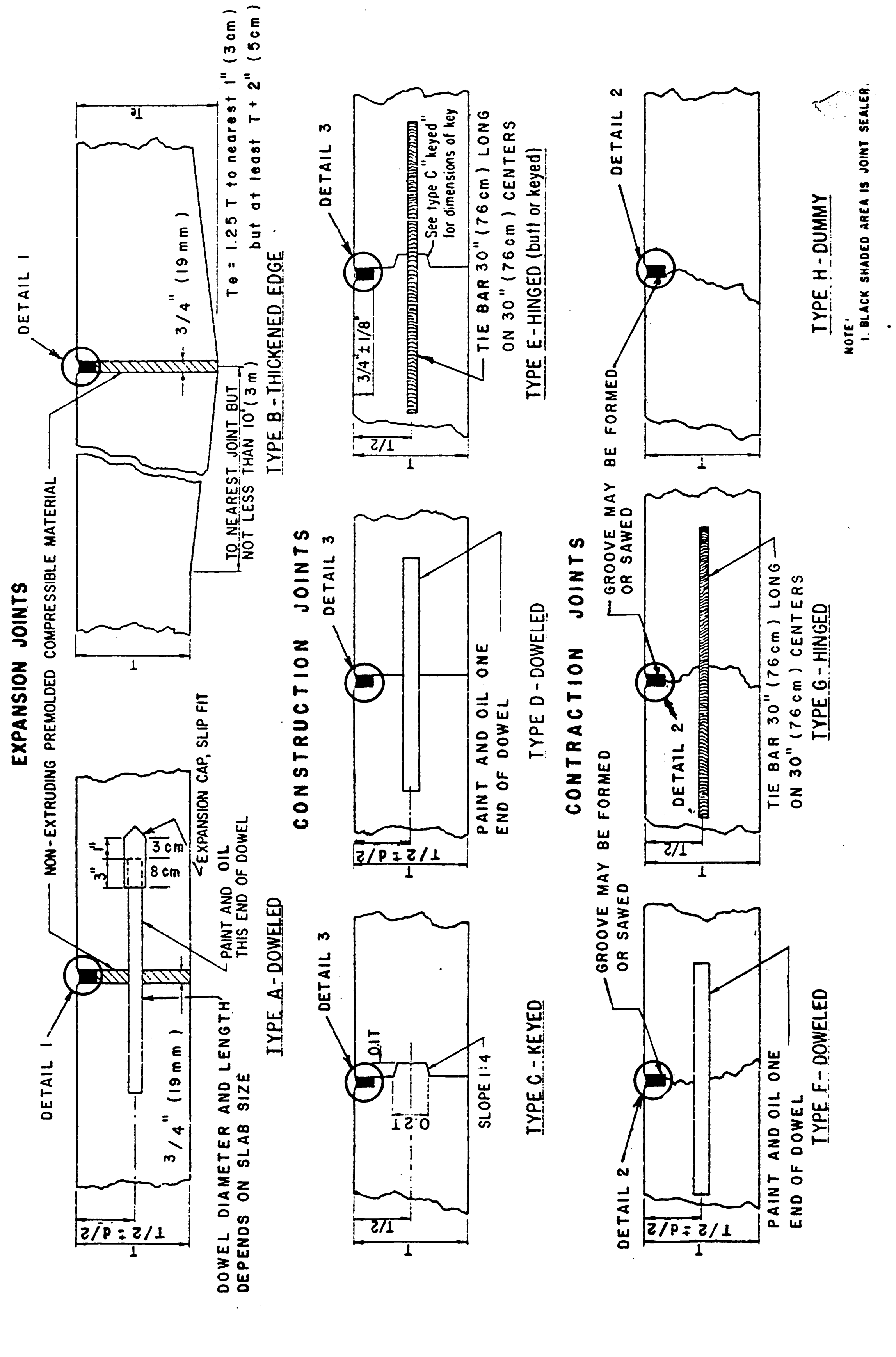
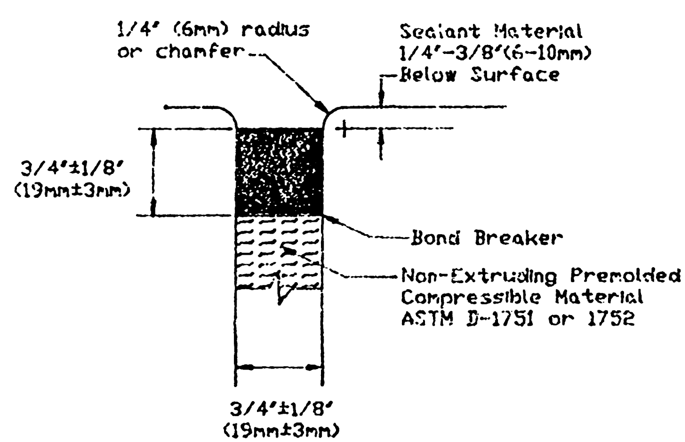
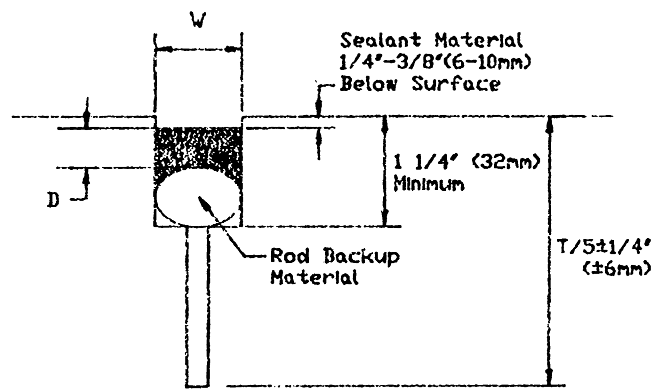


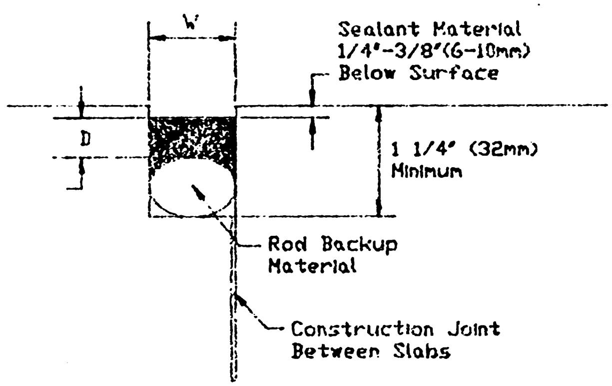
FIGURE 3-42. RIGID PAVEMENT JOINT TYPES AND DETAILS



DETAIL 1
EXPANSION JOINT



DETAIL 2
CONTRACTION JOINT



DETAIL 3
CONSTRUCTION JOINT

Notes:
1. Sealant Reservoir Sized to Provide Proper Shape Factor, W/D, Field Poured and Prefpormed Sealants Require Different Shape Factors for Uptimum Performance.
2. Rod Back-Up Material Must Be Compatible With the Type of Liquid Sealant Used and Sized to Provide the Desired Shape Factor.

FIGURE 3-42. RIGID PAVEMENT JOINT TYPES AND DETAILS

With Stabilized Subbase. Rigid pavements supported on stabilized subbase are subject to higher warping and curling stresses than those supported on unstabilized foundations. When designing a rigid pavement supported on a stabilized subbase a different procedure is recommended to determine joint spacing. Joint spacing should be a function of the radius of relative stiffness of the slab. The joint spacing should be selected such that the ratio of the joint spacing to the radius of relative stiffness is between 4 and 6. The radius of relative stiffness is defined by Westergaard as the stiffness of the slab relative to the stiffness of the foundation. It is determined by the following formula:

$$l = \left(\frac{Eh^3}{12(1-u^2)k}\right)^{\frac{1}{4}}$$

Where:
E = modulus of elasticity of the concrete, usually 4 million psi
h = slab thickness, in.
u = Poisson's ratio for concrete, usually 0.15
k = modulus of subgrade reaction, pci

The radius of relative stiffness has the dimension of length and when calculated in accordance with the above, the units of l are inches.

TABLE 3-11. RECOMMENDED MAXIMUM JOINT SPACINGS RIGID PAVEMENT WITHOUT STABILIZED SUBBASE

Slab Thickness		Transverse		Longitudinal	
Inches	Millimeters	Feet	Meters	Feet	Meters
6	150	12.5	3.8	12.5	3.8
7-9	175-230	15	4.6	15	4.6
9-12	230-305	20	6.1	20	6.1
> 12	> 305	25	7.6	25	7.6

Note: The joint spacings shown in this table are recommended maximum values. Smaller joint spacings should be used if indicated by past experience. Pavements subject to extreme seasonal temperature differentials or extreme temperature differentials during placement may require smaller joint spacings. See also Chapter 5 for light load rigid pavement jointing.

338. SPECIAL JOINTING CONSIDERATIONS. A number of special considerations are required when designing the jointing system for a portland cement concrete pavement. Several considerations are discussed below.

a. Keyed Joints. Keyed construction joints should not be used for slabs less than 9 inches (230 mm) in thickness. Keyed joints in slabs of lesser thickness result in very small keys and key-ways with limited strength.

b. **Jointing Systems for Wide Body Jet Aircraft.** Experience indicates poor performance may result from keyed longitudinal construction joints supported on low-strength foundations when wide body aircraft loadings are encountered. Special jointing recommendations are discussed below.

(1) **Low Strength Foundations.** For foundation moduli of 200 PCI (54 MN/m^3) or less, a doweled or thickened edge construction joint, Type D or B, is recommended. Keyed joints should not be used as poor performance will likely result. In areas of low traffic usage, such as extreme outer lanes of runways and aprons, keyed joints, Type C, may be used.

(2) **Medium Strength Foundations.** For foundation moduli between 200 PCI (54 MN/m^3) and 400 PCI (109 MN/m^3) hinged construction joints, Type E, may be used as well as doweled or thickened edge. The maximum width of pavement which can be tied together depends on several factors such as subgrade frictional restraints, pavement thickness, and climatic conditions. Normally, the maximum width of tied pavement should not exceed 75 feet (23 m). Type C joints may be used in low traffic areas.

(3) **High Strength Foundations.** For foundation moduli of 400 PCI (109 MN/m^3) or greater conventional keyed joints, Type C, may be used regardless of traffic usage. The designer is reminded, however, that the prohibition

against keyed joints in pavements less than 9 inches (230 mm) thick shall still remain in effect.

c. **Future Expansion.** When a runway or taxiway is likely to be extended at some future date, it is recommended that a thickened edge joint be provided at that end of the runway or taxiway. Likewise, if any pavement is to be widened in the future, a key-way or thickened edge should be provided at the appropriate edge.

339. JOINTING STEEL.

a. **Tie Bars.** Tie bars are used across certain longitudinal contraction joints and keyed construction joints to hold the slab faces in close contact. The tie bars themselves do not act as load transfer devices. By preventing wide opening of the joint, load transfer is provided by the keyed joint or by aggregate interlock in the crack below the groove-type joint. Tie bars should be deformed bars conforming to the specifications given in Item P-501. The bars should be 5/8 inch (16 mm) in diameter and 30 inches (760 mm) long and spaced 30 inches (760 mm) on center.

b. **Dowels.** Dowels are used at joints to provide for transfer of load across the joint and to prevent relative vertical displacement of adjacent slab ends. Dowels permit longitudinal movement of adjacent slabs.

(1) **Where Used.** Provision for load transfer by dowels is provided at all transverse expansion joints and all butt-type construction joints. Dowels for contraction joints should be provided at least three joints from a free edge. Contraction joints in the interior of the pavement may be the dummy groove type.

(2) **Size Length and Spacing.** Dowels should be sized such that they will resist the shearing and bending stresses produced by the loads on the pavement. They should be of such length and spacing that the bearing stresses exerted on the concrete will not cause failure of the concrete slab. Table 3-12 indicates the dowel dimensions and spacing for various pavement thicknesses.

TABLE 3-12. DIMENSIONS AND SPACING OF STEEL DOWELS

Thickness of Slab	Diameter	Length	Spacing
6-7 in (150-180 mm)	3/4 in (20 mm)	18 in (460 mm)	12 in (305 mm)
8-12 in (210-305 mm)	1 in (25 mm)	19 in (480 mm)	12 in (305 mm)
13-16 in (330-405 mm)	1 1/4 in[1] (30 mm)	20 in (510 mm)	15 in (380 mm)
17-20 in (430-510 mm)	1 1/2 in[1] (40 mm)	20 in (510 mm)	18 in (460 mm)
21-24 in (535-610 mm)	2 in[1] (50 mm)	24 in (610 mm)	18 in (460 mm)

[1]Dowels noted may be a solid bar or high-strength pipe. High-strength pipe dowels must be plugged on each end with a tight-fitting plastic cap or with bituminous or mortar mix.

(3) **Dowel Positioning.** The alignment and elevation of dowels is extremely important in obtaining a satisfactory joint. Transverse dowels will require the use of a fixture, usually a wire cage or basket firmly anchored to the subbase, to hold the dowels in position. During the concrete placement operations, it is advisable to place plastic concrete directly on the dowel assembly immediately prior to passage of the paver to prevent displacement of the assembly by the paving equipment. Some paving machines have a dowel placer which can also be used to accurately position dowels.

340. JOINT SEALANTS AND FILLERS. Sealants are used in all joints to prevent the ingress of water and foreign material in the joint. Premolded compressible fillers are used in expansion joints to permit expansion of the slabs. Joint sealants are applied above the filler in expansion joints to prevent infiltration of water and foreign material. In areas subject to fuel spillage, fuel resistant sealants should be used. Specifications for joint sealants are given in Item P-605.

341. JOINT LAYOUT. Pavement joint layout is a matter of selecting the proper joint types and locations so that the joints can perform their intended function. Construction considerations are also vitally important in determining the joint layout pattern. Paving lane widths will often dictate how the pavement should be jointed. Generally speaking it is more economical to keep the number of passes of the paving train to a minimum while maintaining proper joint function. Figure 3-43 shows a typical jointing plan for a runway end, parallel taxiway and connector. It is impossible to illustrate all of the variations which can occur at pavement intersections. Reference 8 in Appendix 4 contains further information on jointing patterns. Two important considerations in designing joint layouts for intersections are isolation joints and odd-shaped shapes. More discussion on these follows:

a. **Isolation Joints**. Two intersecting pavements such as a taxiway and runway should be isolated to allow the pavements to move independently. Isolation can best be accomplished by using a Type B expansion joint between the two pavements. The expansion joint should be positioned such that the two pavements can expand and contract independently; normally this can be accomplished by using a Type B expansion joint where the two pavements abut. One isolation joint is normally sufficient to allow independent movement.

b. **Odd-Shaped Slabs**. Cracks tend to form in odd-shaped slabs; therefore, it is normally good practice to maintain sections which are nearly square or rectangular in shape. Pavement intersection which involve fillets are difficult to design without a few odd-shaped slabs. In instances where odd-shaped slabs cannot be avoided, steel reinforcement is recommended. Steel reinforcement should consist of 0.050% steel in both directions in slabs where the length-to-width ratio exceeds 1.25 or in slabs which are not rectangular in shape. The steel reinforcement should be placed in accordance with the recommendations given in Paragraph 341, Reinforced Concrete Pavement. Fillets may also be defined by constructing slabs to the normal, full dimensions and painting out the unused portion of the slab with bitumen.

342. REINFORCED CONCRETE PAVEMENT. The main benefit of steel reinforcing is that, although it does not prevent cracking, it keeps the cracks that form tightly closed so that the interlock of the irregular faces provides structural integrity and usually maintains pavement performance. By holding the cracks tightly closed, the steel minimizes the infiltration of debris into the cracks. The thickness requirements for reinforced concrete pavements are the same as plain concrete and are determined from the appropriate design curves, Figures 3-17 through 3-41. Steel reinforcement allows longer joint spacings, thus the cost benefits associated with fewer joints must be considered in the decision to use plain or reinforced concrete pavement.

343. TYPE AND SPACING OF REINFORCEMENT. Reinforcement may be either welded wire fabric or bar mats installed with end and side laps to provide complete reinforcement throughout the slab panel. End laps should be a minimum of 12 inches (305 mm) but not less than 30 times the diameter of the longitudinal wire or bar. Side laps should be a minimum of 6 inches (150 mm) but not less than 20 times the diameter of the transverse wire or bar. End and side clearances should be a maximum of 6 inches (150 mm) and a minimum of 2 inches (50 mm) to allow for nearly complete reinforcement and yet achieve adequate concrete cover. Longitudinal members should be spaced not less than 4 inches (100 mm) nor more than 12 inches (305 mm) apart; transverse members should be spaced not less than 4 inches (100 mm) nor more than 24 inches (610 mm) apart.

344. AMOUNT OF REINFORCEMENT.

a. The steel area required for a reinforced concrete pavement is determined from the subgrade drag formula and the coefficient of friction formula combined. The resultant formula is expressed as follows:

$$A_s = 3.7 \frac{L^2 t}{f_s}$$

where:

A_s	=	area of steel per foot of width or length, square inches
L	=	length or width of slab, feet
t	=	thickness of slab, inches
f_s	=	allowable tensile stress in steel, psi

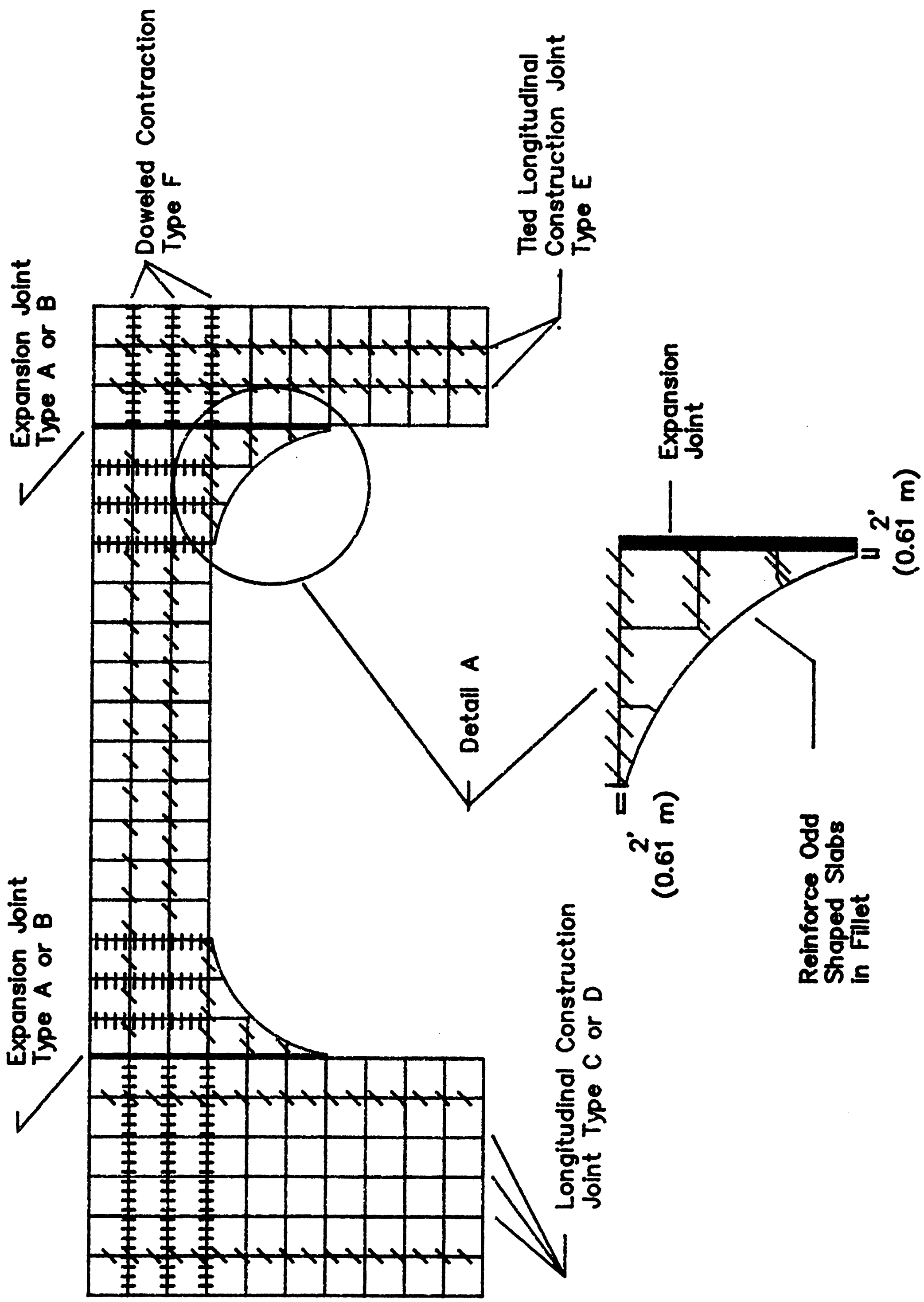


FIGURE 3-43. TYPICAL JOINT LAYOUT PATTERN FOR RUNWAY, PARALLEL; TAXIWAY AND CONNECTOR

NOTE: To determine the area of steel in metric units:

L should be expressed in meters
t should be expressed in millimeters
f_s should be expressed in Mega newtons per square meter
The constant 3.7 should be changed to 0.64.
A_s will then be in terms of square centimeters per meter.

b. In this formula the slab weight is assumed to be 12.5 pounds per square foot, per inch of thickness (23.6 MN/m^2). The allowable tensile stress in steel will vary with the type and grade of steel. It is recommended that allowable tensile stress be taken as two-thirds of the yield strength of the steel. Based on current specifications the yield strengths and corresponding design stresses (fs) are as listed in Table 3-13.

c. The minimum percentage of steel reinforcement should be 0.05%. The percentage of steel is computed by dividing the area of steel, As, by the area of concrete per unit of length (or width) and multiplying by 100. The minimum percentage of steel considered the least amount of steel which can be economically placed is 0.05%. Steel reinforcement allow larger slab sizes and thus decreases the number of transverse contraction joints. The costs associated with providing a reinforced pavement must be compared with the savings realized in eliminating some of the transverse contraction joints to determine the most economical steel percentage. The maximum allowable slab length regardless of steel percentage is 75 feet (23 m).

TABLE 3-13. YIELD STRENGTHS OF VARIOUS GRADES OF REINFORCING STEEL

ASTM Designation	Type & Grade of Steel	Yield Strength psi (MN/m^2)	FS psi (MN/m^2)
A 615	Deformed Billet Steel, Grade 40	40,000 (300)	27,000 (200)
A 616	Deformed Rail Steel, Grade 50	50,000 (370)	33,000 (240)
A 616	Deformed Rail Steel, Grade 60	60,000 (440)	40,000 (300)
A 615	Deformed Billet Steel, Grade 60	60,000 (440)	40,000 (300)
A 185	Cold Drawn Welded Steel Wire Fabric	65,000 (480)	43,000 (320)
A 497	Cold Drawn Welded Deformed Steel Wire	70,000 (520)	47,000 (350)

TABLE 3-14. DIMENSIONS AND UNIT WEIGHTS OF DEFORMED STEEL REINFORCING BARS

NOMINAL DIMENSIONS

Number	Diameter in. (mm)	Area in.^{2} (cm^2)	Perimeter in. (cm)	Unit Weight lbs./ft. (kg/m)
3	0.375 (9.5)	0.11 (0.71)	1.178 (3.0)	0.376 (0.56)
4	0.500 (12.7)	0.20 (1.29)	1.571 (4.0)	0.668 (1.00)
5	0.625 (15.9)	0.31 (2.00)	1.963 (5.0)	1.043 (1.57)
6	0.750 (19.1)	0.44 (2.84)	2.356 (6.0)	1.502 (2.26)
7	0.875 (22.2)	0.60 (3.86)	2.749 (7.0)	2.044 (3.07)

345. DIMENSIONS AND WEIGHTS OF REINFORCEMENT. Dimensions and unit weights of standard deformed reinforcing bars are given in Table 3-14, and wire size number, diameters, areas, and weights of wires used in welded wire fabric are given in Table 3-15.

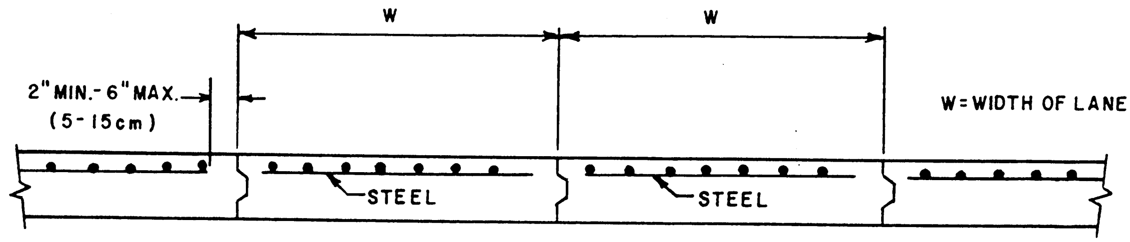
346. WELDED WIRE FABRIC. The use of welded wire fabric requires some special design considerations to achieve the most economical design. The use of smooth welded wire fabric or deformed welded wire fabric is the option of the designer. The choice should be based on the difference in allowable design stresses, the availability of the desired sizes (smooth wire fabric is available in a wider range of sizes), and the costs associated with each style of fabric. It is recommended that the minimum size of longitudinal wire by W5 or D5. The minimum transverse wire should be no smaller than W4 or D4. In addition, should calculated area of longitudinal steel be less than 0.05 percent of the cross-sectional area of slab, the size and spacing of the steel members (bars or wire) should be determined on the premise that the minimum area should not be less than 0.05 percent. This percentage applies in the case of steel having a yield strength of 65,000 PSI (480 MN/m^2). If lower grades are used, the percentage should be revised proportionately upward. For example, Table 3-15 shows that W10 wires, spaced 10 inches (255 mm) apart, furnish an area of 0.12 square inches (77 mm^2) which satisfies the requirement for pavements up to 20 inches (510 mm) thick. Sizing of individual sheets of welded wire fabric is also important in providing an economical design. Not all fabricators supply all wire sizes in all spacings. While nearly any fabric style can be produced on special order, it is generally more economical to specify a standard production configuration. Sheet and roll widths in excess of 8 feet (2.5 m) can result in higher shipping costs.

TABLE 3-15. SECTIONAL AREAS OF WELDED FABRIC

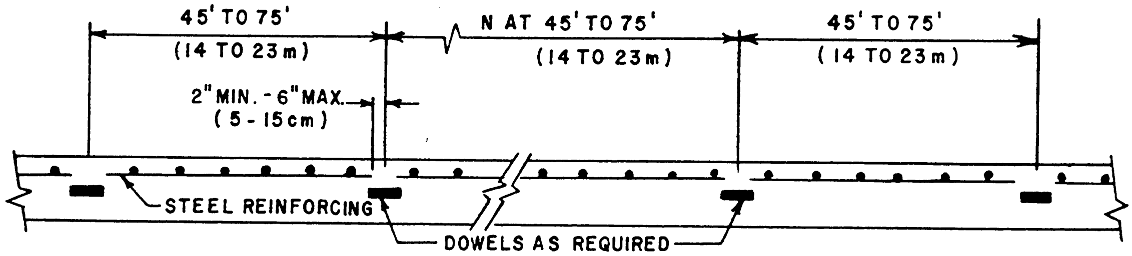
Wire Size Smooth	Number Deformed	Nominal Diameter Inches	Nominal Weight lbs./lin.ft.	Center-to-Center Spacing 4"	6"	8"	10"	12"
W31	D31	0.628	1.054	.93	.62	.465	.372	.31
W30	D30	0.618	1.020	.90	.60	.45	.36	.30
W28	D28	0.597	.952	.84	.56	.42	.336	.28
W26	D26	0.575	.934	.78	.52	.39	.312	.26
W24	D24	0.553	.816	.72	.48	.36	.288	.24
W22	D22	0.529	.748	.66	.44	.33	.264	.22
W20	D20	0.504	.680	.60	.40	.30	.24	.20
W18	D18	0.478	.612	.54	.36	.27	.216	.18
W16	D16	0.451	.544	.48	.32	.24	.192	.16
W14	D14	0.422	.476	.42	.28	.21	.168	.14
W12	D12	0.390	.408	.36	.24	.18	.144	.12
W11	D11	0.374	.374	.33	.22	.165	.132	.11
W10.5		0.366	.357	.315	.21	.157	.126	.105
W10	D10	0.356	.340	.30	.20	.15	.12	.10
W9.5		0.348	.323	.285	.19	.142	.114	.095
W9	D9	0.338	.306	.27	.18	.135	.108	.09
W8.5		0.329	.289	.255	.17	.127	.102	.085
W8	D8	0.319	.272	.24	.16	.12	.096	.08
W7.5		0.309	.255	.225	.15	.112	.09	.075
W7	D7	0.298	.238	.21	.14	.105	.084	.07
W6.5		0.288	.221	.195	.13	.097	.078	.065
W6	D6	0.276	.204	.18	.12	.09	.072	.06
W5.5		0.264	.187	.165	.11	.082	.066	.055
W5	D5	0.252	.170	.15	.10	.075	.06	.05
W4.5		0.240	.153	.135	.09	.067	.054	.045
W4	D4	0.225	.136	.12	.08	.06	.048	.04

Note: 1 inch = 2.54 cm
1 lb./lin. ft. = 1.5 kg/m

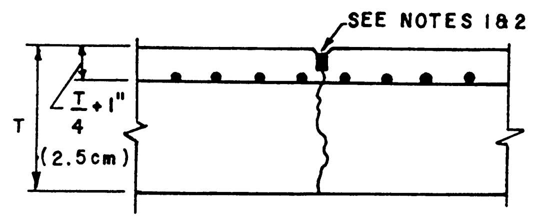
347. JOINTING OF REINFORCED PAVEMENTS. Contraction joints in reinforced pavements may be spaced up to 75 feet (23 m) apart, and all joints should be provided with load transfer devices as shown in Figure 3-44. Also, this figure presents other reinforcement details such as clearance at joints and edges of pavement and depth below the surface. The longer joint spacing allowed with reinforced pavements will result in larger joint openings. The joints must be sealed carefully to accommodate the larger movements at the joints.



TRANSVERSE CROSS SECTION OF PAVING LANES



LONGITUDINAL CROSS SECTION



LONGITUDINAL CONTRACTION JOINT
(SEE NOTE 3)

NOTES:

1. SEE FIGURES 3-30 & 3-31 FOR GROOVE DETAILS
2. JOINT DETAILS ARE SIMILAR TO FIGURES 3-30 & 3-31 EXCEPT FOR STEEL REINFORCING.
3. USE THIS JOINT WHEN THE SLAB THICKNESS IS 10 INCHES (25cm) OR LESS AND PAVING EXCEEDS $12\frac{1}{2}$ FEET (4m).

FIGURE 3-44 JOINTING OF REINFORCED RIGID PAVEMENTS

348. CONTINUOUSLY REINFORCED CONCRETE PAVEMENT. A continuously reinforce concrete pavement (CRCP) is a portland cement concrete pavement with continuous longitudinal steel reinforcement and no intermediate transverse expansion or contraction joints. Continuously reinforced concrete pavements normally contain from 0.5 to 1.0 percent longitudinal steel reinforcement. The main advantage of continuously reinforced concrete pavement is the elimination of transverse joints which are costly to construct, require periodic resealing, and are often a source of maintenance problems. Continuously reinforced concrete pavements usually provide a very smooth riding surface. A properly designed CRCP will develop random transverse cracks at 2 to 10 feet (0.6 to 3 m) intervals. The resultant pavement is composed of a series of articulated short slabs held tightly together by the longitudinal reinforcing steel. A high degree of shear transfer across the cracks can be achieved because the cracks are held tightly closed.

a. Foundation Support. The reinforcing steel in a CRCP provides continuity of load transfer however good uniform foundation support must still be provided for satisfactory performance. The embankment and subbase requirements given earlier in this Chapter for plain concrete pavements also apply to CRCP.

b. Thickness Design. The thickness requirements for CRCP are the same as plain concrete and are determined from the appropriate design curves, Figures 3-17 through 3-41. Design inputs are the same for concrete strength, foundation strength, aircraft weight and departure level.

c. Longitudinal Steel Design. The design of steel reinforcement for CRCP is critical to providing a satisfactory pavement. The steel percentage must be properly selected to provide optimum crack spacing and crack width. Crack widths must be small to provide a high degree of shear transfer across the crack and to prevent the ingress of water through the crack. The design of longitudinal steel reinforcement must satisfy three conditions. The maximum steel percentage determined by any of the three following requirements should be selected as the design value. In no case should the longitudinal steel percentage be less than 0.5 percent.

(1) Steel to Resist Subgrade Restraint. The longitudinal steel reinforcement required to resist the forces generated by the frictional restraint between the CRCP and the subbase should be determined by using the nomograph shown on Figure 3-45. Use of the nomograph requires three parameters: allowable working stress for steel, tensile strength of concrete and a friction factor for the subbase. The recommended working stress for steel is 75 percent of the specified minimum yield strength. The tensile strength of concrete may be estimated as 67 percent of the flexural strength. The recommended friction factor for stabilized subbase is 1.8. While not recommended as subbase for CRCP, friction factors for unbound fine-grained soils and coarse-grained soils are usually assumed to be 1.0 and 1.5 respectively.

(2) Steel to Resist Temperature Effects. The longitudinal steel reinforcement must be capable of withstanding the forces generated by the expansion and contraction of the pavement due to temperature changes. The following formula is used to compute the temperature reinforcement requirements.

$$P_s = \frac{50 f_t}{f_s - 195T}$$

where:

P_s = steel reinforcement in percent
f_t = tensile strength of concrete
f_s = working stress for steel usually taken as 75% of specified minimum yield strength
T = maximum seasonal temperature differential for pavement in degrees Fahrenheit

Reinforcing steel should be specified on the basis of minimum yield strength. All deformed reinforcing steel bars should conform to ASTM A 615, A 616 or A 617. Deformed welded wire fabric should conform to ASTM A 497.

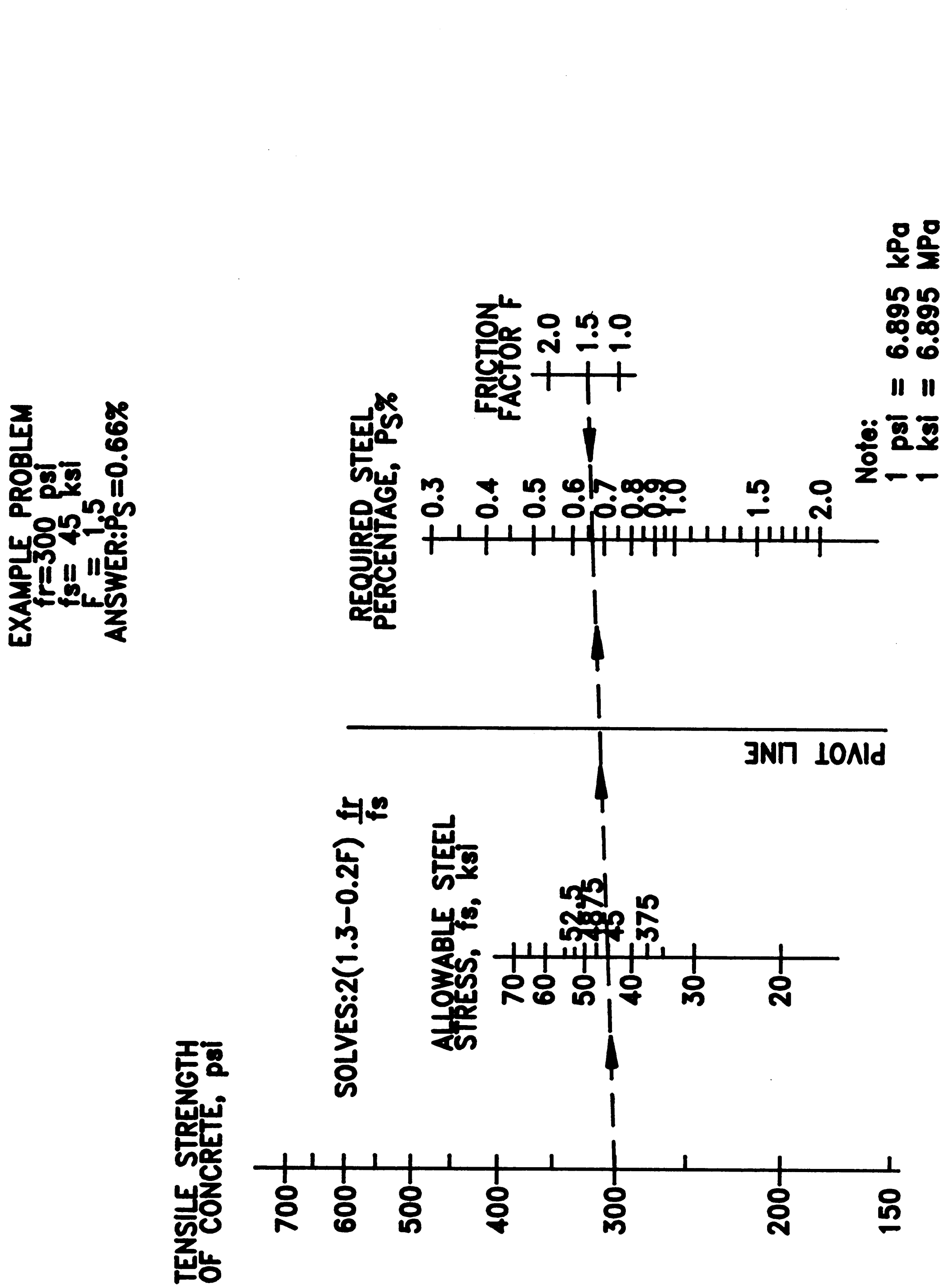


FIGURE 3-45. CONTINUOUSLY REINFORCING CONCRETE PAVEMENT - LONGITUDINAL STEEL REINFORCEMENT

(3) **Concrete to Steel Strength Ratio.** The third consideration in selecting the amount of longitudinal steel reinforcement is the ratio of concrete tensile strength to the specified minimum yield strength of steel. The steel percentage is obtained by multiplying the ratio of the concrete strength to the yield strength of steel by 100.

$$P_s = \frac{100 f_t}{f_y}$$

where:
P_s = steel reinforcement in percent
f_t = tensile strength of concrete
f_y = minimum yield strength of steel

d. Transverse Steel Design. Transverse steel reinforcement is recommended for CRC airport pavements to control "chance" longitudinal cracks which sometimes form. It is also aids in construction by supporting and maintaining longitudinal steel reinforcement spacing. A nomograph for determining transverse steel requirements is shown in Figure 3-46.

e. Steel Detailing. Longitudinal steel reinforcement should be located at mid depth of the slab or slightly above. Transverse steel may be located either above or below the longitudinal steel. A minimum concrete cover of 3 inches (75 mm) should be maintained over all steel reinforcement. Longitudinal steel spacing should be 6 to 12 inches (150 to 310 mm). Transverse steel should be spaced at 12 inches (310 mm) or greater. The recommended overlap for splicing of reinforcing bars is 25 diameters of 16 inches (405 mm), whichever is greater. The recommended overlap for splicing deformed welded wire fabric is 32 diameters or 16 inches (405 mm), whichever is greater. When splicing longitudinal steel bar reinforcing it is recommended that the lap splices be made on a 60 degree skew from centerline or staggered such that not more than 1/3 of the bars are spliced on the same transverse plane.

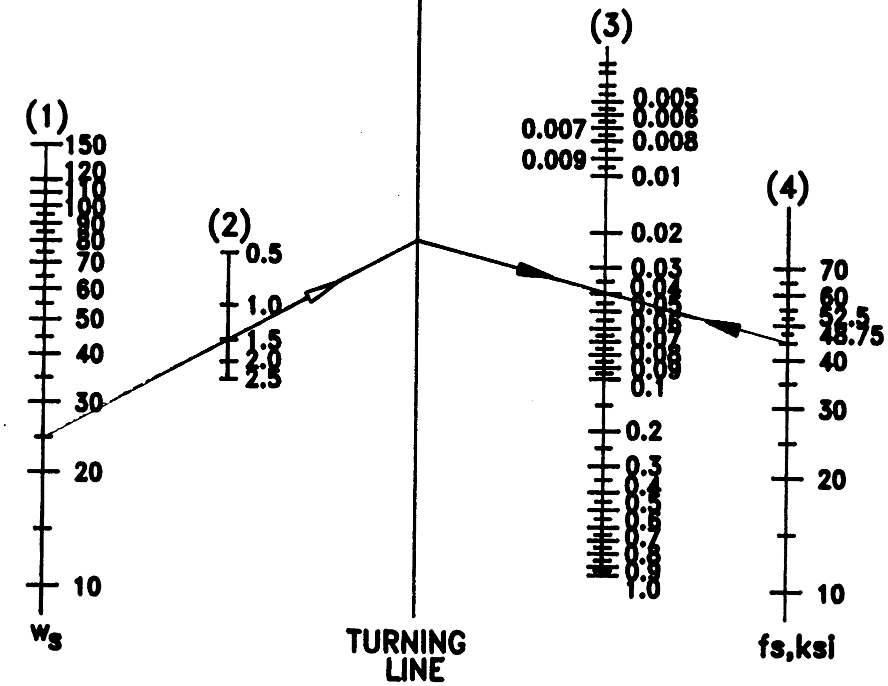
349. CRCP JOINTING. Even though transverse contraction joints can be eliminated with CRCP, some joints will be needed to accommodate construction and to control warping stresses. The two types of joints are discussed below:

a. Construction Joints. Two types of construction joints are necessary for CRCP. Because pavements are constructed in multiple lanes, a longitudinal construction joint is required between lanes. A transverse construction joint must be provided where paving ends and begins, such as at the finish of a day's paving and the start of the next day's paving. Typical construction joint details are shown in Figure 3-47.

b. Warping Joints. Warping joints or hinged joints are needed when paving lane width exceeds the recommended maximum longitudinal joint spacings shown in Table 3-11. Transverse steel is carried through the joint to provide continuity and positive aggregate interlock across the joint. Since carrying the steel through the joint eliminates any expansion or contraction capacity, the maximum width of tied pavement should not exceed 75 feet (23 m), see paragraph 337.b.(2). Typical warping joint details are shown in Figure 3-47.

350. CRCP TERMINAL TREATMENT. Since long slabs of CRCP are constructed with no transverse joints, provisions must be made to either restrain or accommodate end movements wherever the CRCP abuts other pavements or structures. Rather large end movements, up to 2 inches (50 mm), are experienced with CRCP due to thermal expansion and contraction. End movement is normally not a problem except where CRCP abuts another pavement or structure. Experience with highway CRCP shows that attempts to restrain end movement have not been too successful. More favorable results are achieved where end movement is accommodated rather than restrained. Joints designed to accommodate large movements are required where CRCP intersects other pavements or abuts another structures. Failure to do so may result in damage to the CRCP, pavement or other structure. Wide flange beam type joints or "finger" type expansion joints can accommodate the movements. The wide flange beam type joint is recommended due to its relatively lower costs. A sketch of the wide flange beam joint is shown on Figure 3-48.

SOLVES: $P = \dfrac{W_s F}{2 f_s} \times 100$



EXAMPLE PROBLEM
W_s =25 FT
F =1.5
f_s=45,000psi
ANSWER:P =0.04%

WHERE:
P_s =REQUIRED STEEL PERCENTAGE, %
W_s =WIDTH OF SLAB, FEET
F =FRICTION FACTOR OF SUBGRADE, SUBBASE, OR STRESS-RELEIVING LAYER
f_s=ALLOWABLE WORKING STRESS IN STEEL, PSI (0.75 OF YIELD STRENGTH RECOMMENDED, THE EQUIVALENT OF SAFETY FACTOR OF 1.33)

Note:
1 psi = 6.895 kPa
1 ksi = 6.895 MPa
1 ft = 0.3048 m

FIGURE 3-46. CONTINUOUSLY REINFORCING CONCRETE PAVEMENT - TRANSVERSE STEEL REINFORCEMENT

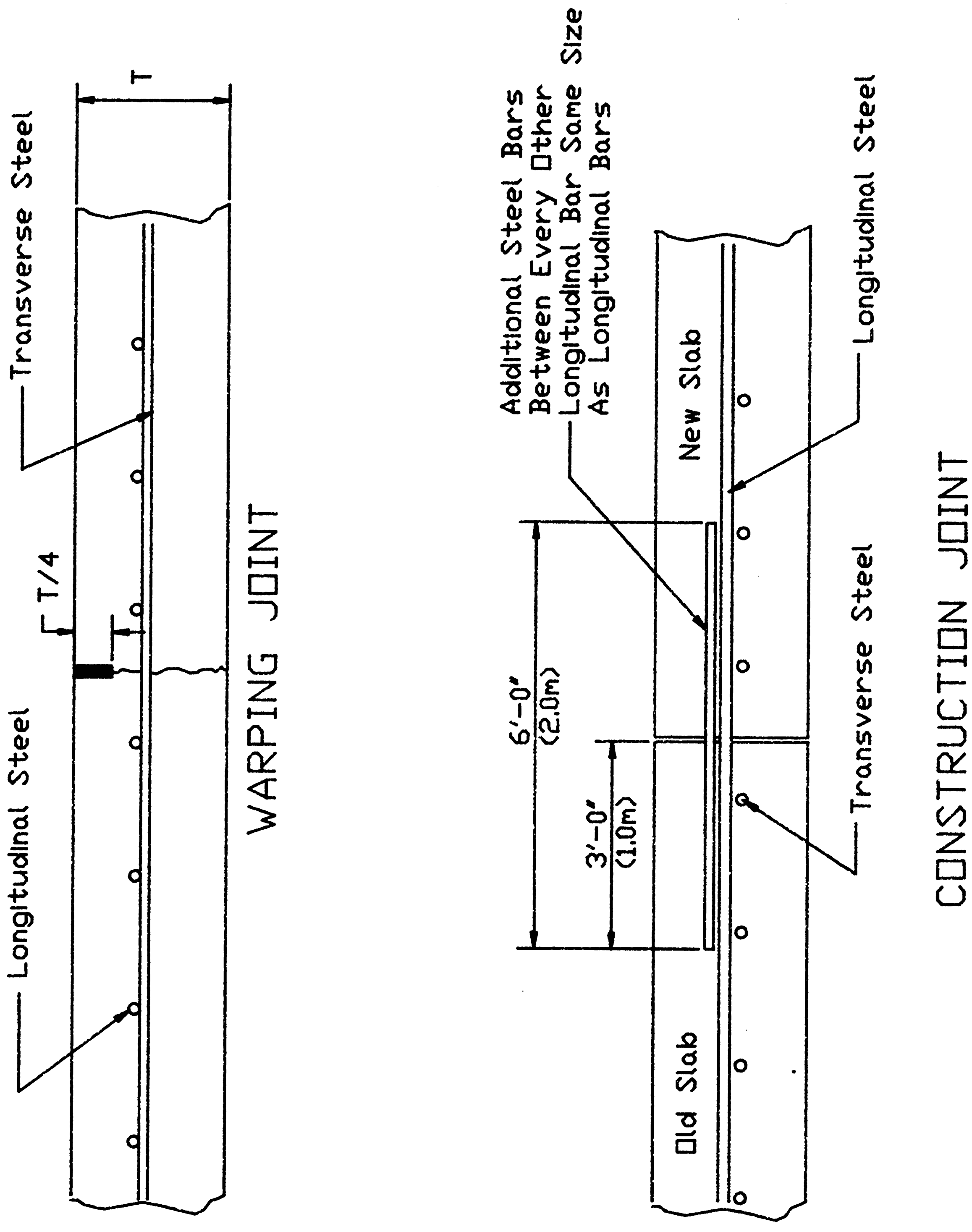


FIGURE 3-47. CONTINUOUSLY REINFORCED CONCRETE PAVEMENT - JOINTING DETAILS

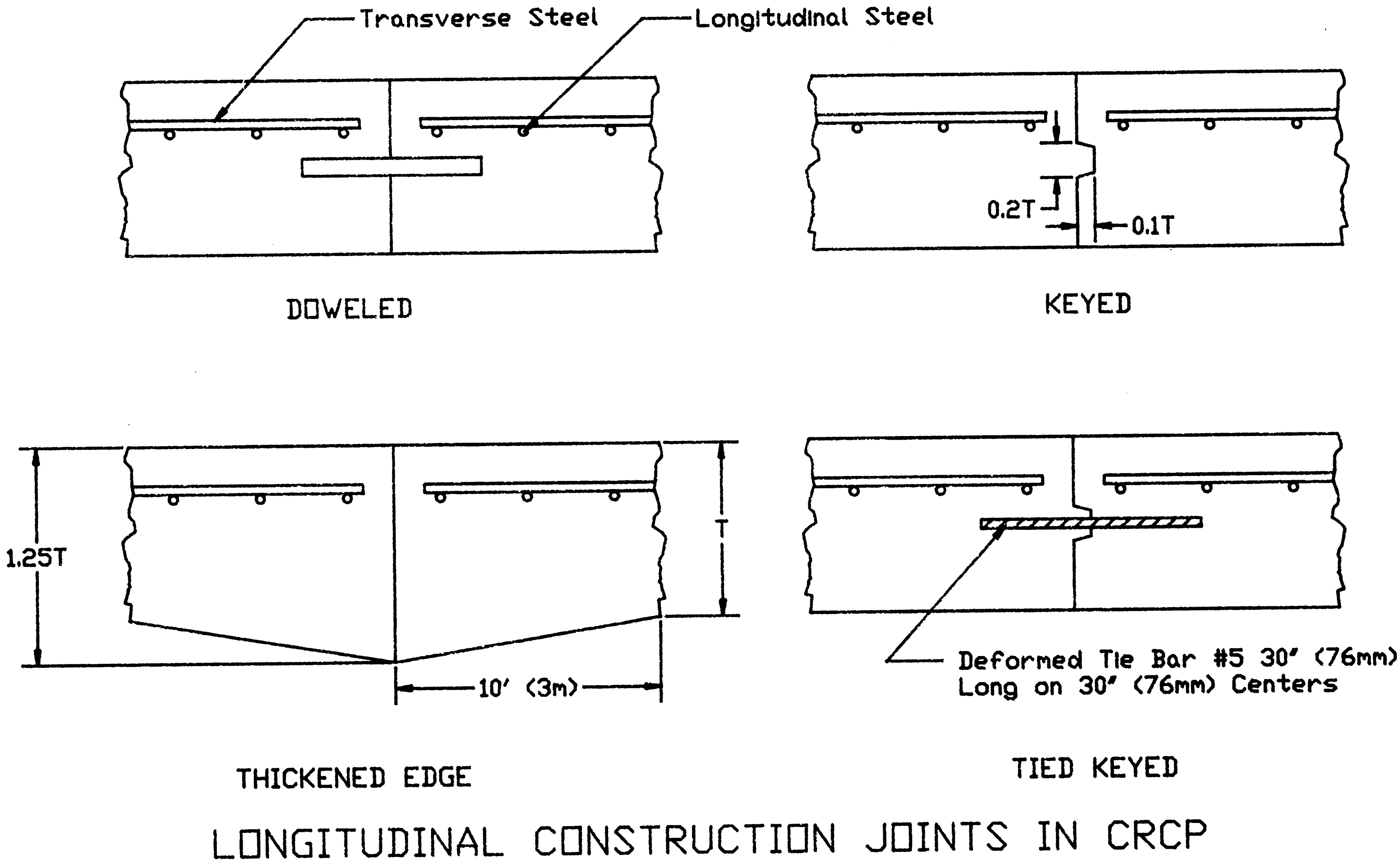
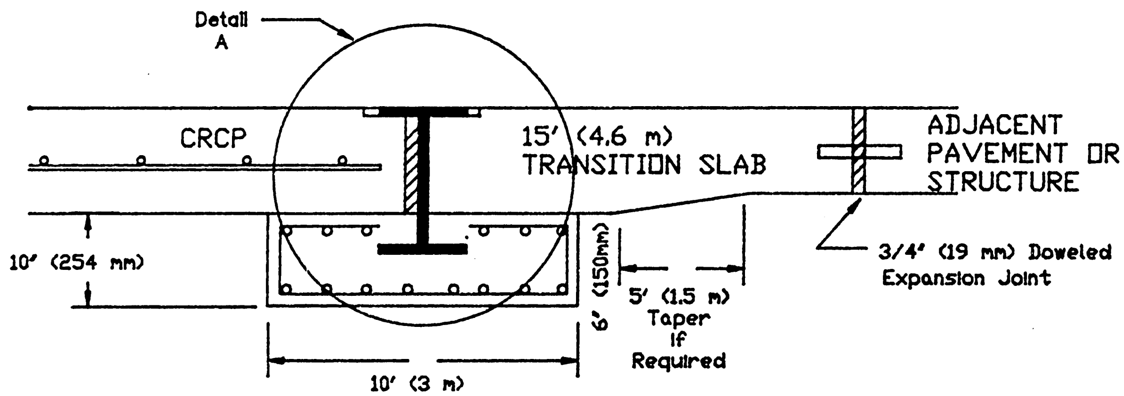


FIGURE 3-47. CONTINUOUSLY REINFORCED CONCRETE PAVEMENT - JOINTING DETAILS



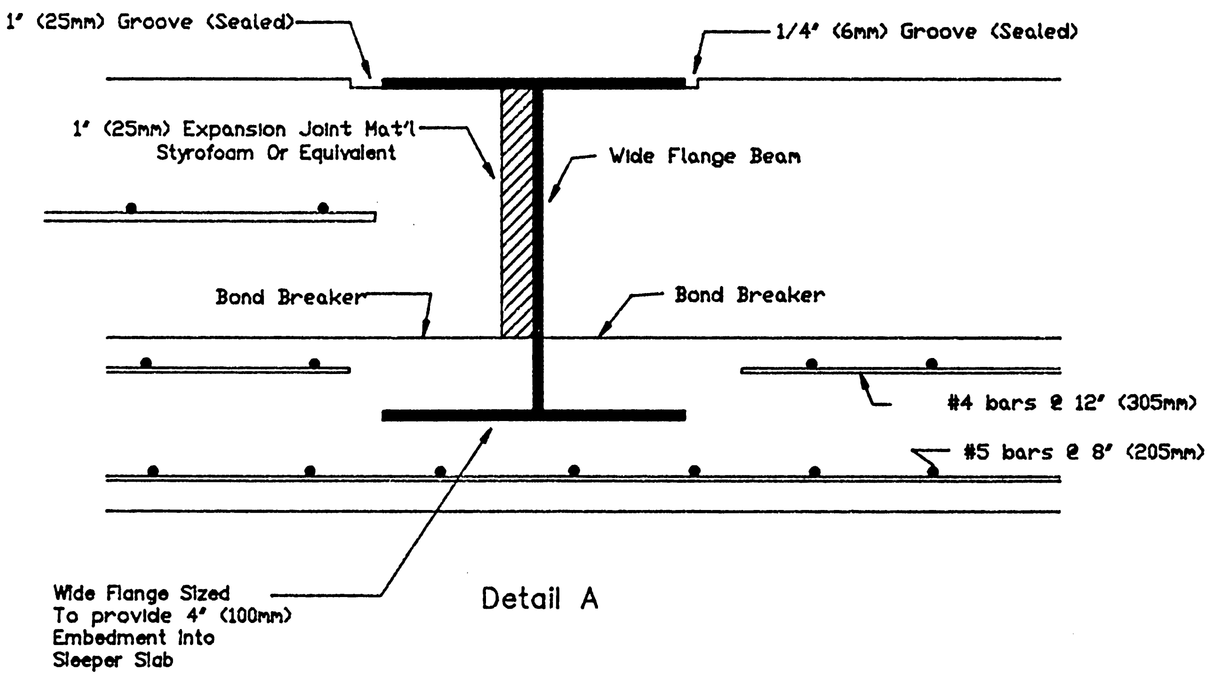


FIGURE 3-48. CONTINUOUSLY REINFORCED CONCRETE PAVEMENT - WIDE FLANGE BEAM TERMINAL JOINT

351. CRCP DESIGN EXAMPLE. An example design for CRCP is given below. Assume a CRCP is to be designed to serve the following conditions:

a. **Design Aircraft** - DC-10-10 with a gross weight of 400,000 lbs (182,000 kg)

b. **Foundation Modulus** - 400 pci (109 MN/m^3)

c. **Concrete Flexural Strength** - 600 psi (4.2 MPa)

d. **Annual Departures** - 3000

e. **Minimum Specified Yield Strength of Steel** - 60,000 psi (414 MPa) (Longitudinal and Transverse)

f. **Paving Lane Width** - 25 feet (7.6 m)

g. **Cement Stabilized Subbase**

h. **Seasonal Temperature Differential** - 100°F (38°C)

(1) **Slab Thickness.** Enter the design curve for DC-1010 aircraft, Figure 3-33, with the parameters assumed above and read a pavement thickness of 12.2 inches (310 mm). This thickness would be rounded down to the next full inch or 12.0 inches (305 mm).

(2) **Steel Design.** The longitudinal reinforcing steel would be determined as described in paragraph c above:

(i) **Subgrade Restraint.** Using the nomograph in Figure 3-45 the longitudinal steel required to with the forces generated by subgrade restraint is found to be 0.83 percent. With the following inputs:
Working stress = 75% x 60,000 = 45,000 psi (310 MPa)
Friction factor = 1.8
Tensile strength of concrete = 67% of 600 = 400 psi (2.8 MPa)

(ii) **Temperature Effects.** The steel required to withstand the forces generated by seasonal temperature changes is computed using the formula given in paragraph 348.c.(2).

$$Ps = \frac{50\times}{45,000-195\times} = 0.78\%$$

(iii) **Concrete to Steel Strength Ratio.** The strength ratio between the concrete and steel is computed by the procedure given in paragraph 348.c.(3).

$$Ps = \frac{400\times}{60,000} = 0.67\%$$

(iv) **Transverse Steel.** The transverse reinforcing steel percentage would be determined using Figure 3-46. This will yield a transverse steel requirement of 0.055%

(v) **Final Design.** The final design would be a 12 inch (305 mm) thick concrete slab. Since the steel percentage necessary to satisfy the subgrade restraint condition is the largest steel percentage for longitudinal reinforcement, the value of 0.83 percent would be selected for design. The transverse steel requirement is 0.055%. The longitudinal steel requirement can be satisfied by using #7 reinforcing bars spaced at 6 inches (150 mm). The transverse steel requirement can be met by using #4 bars on 30 inch (760 mm) centers.

352. PRESTRESSED CONCRETE PAVEMENT. Prestressed concrete pavements have been used in airport applications in Europe and to a limited extent in the United States. Prestressed concrete airport pavements are usually post-tensioned with high strength steel strands. These pavements are usually considerably thinner than plain, jointed reinforced, or continuously reinforced concrete pavements yet provide high load carrying capacity. Slab lengths on the order of 400 to 500 feet (120 to 150 m) are generally used. A design procedure for prestressed airport pavements was developed under an FAA research effort and is reported in Research Report Number FAA-RD-74-34, Volume II. Use of prestressed concrete airport pavements on Federally assisted projects will require FAA approval on a case by case basis.

CHAPTER 4. AIRPORT PAVEMENT OVERLAYS AND RECONSTRUCTION

400. GENERAL. Airport pavement overlays or reconstruction may be required for a variety of reasons. A pavement may require an overlay or reconstruction because the original pavement has served its design life and it is simply "worn out." A pavement may have been damaged by overloading in such a way that it cannot be economically maintained at a serviceable level. Similarly, a pavement in good condition may require strengthening to serve heavier aircraft than those for which the pavement was originally designed. Generally, airport pavement overlays consist of either portland cement concrete or hot mix asphalt concrete. Techniques and equipment are now available to recycle old pavement materials into reconstructed sections. Pavements which are severely distressed in the center portions can sometimes be economically reconstructed by building a keel section using recycled materials. Use of this method of reconstruction is essentially the same as building a new pavement.

401. CONDITION OF EXISTING PAVEMENT. Assessment of the condition of the existing pavement is one of the most important and difficult steps in design of a reconstruction or overlay project. Determination of the properties of the existing pavement should include the thickness, condition and strength of each layer, the subgrade soil classification, and some estimate of foundation strength (CBR or subgrade modulus). An assessment of the structural integrity of the existing pavement is necessary. Failed areas in the existing pavement should be carefully studied to determine the probable cause of failure. Subsurface drainage conditions should be assessed carefully and corrected if found to be deficient. In some instances subsurface drainage corrections are best performed through reconstruction. Overlaying an existing pavement without correcting poor subsurface drainage will usually result in poor overlay performance. A valuable technique for assessing the condition of the existing pavement is nondestructive pavement testing (NDT). See Appendix 3. NDT can be used to estimate foundation strength, measure joint condition, and possibly detect voids in existing pavements.

402. MATERIAL SELECTION CONSIDERATIONS. Criteria are presented in this circular for both hot mix asphalt and concrete reconstruction or overlays. The selection of the material type should be made after careful consideration of many factors. The designer should consider the total life cycle cost of the reconstructed or overlay pavement. (see DOT/FAA/RD-81/78, Appendix 4). Life cycle costs should include initial construction and maintenance costs over the design life of the pavement. Other considerations such as allowable down time of the pavement and availability of alternate pavements to use during construction will have a significant impact on the material selected.

403. OVERLAY DESIGN. The remainder of this chapter is devoted to the design of overlay pavements. As previously mentioned, the design of reconstructed pavements is essentially the same as for new construction.

a. **Typical Overlay Cross Sections and Definitions.** Typical overlay pavement cross sections are shown in Figure 4-1. Definitions applicable to overlay pavements are as follows:

(1) **Overlay Pavement.** Pavement which is constructed on top of an existing pavement.
(2) **Hot Mix Asphalt Overlay.** Hot mix asphalt pavement placed on an existing pavement.
(3) **Concrete Overlay.** Portland cement concrete pavement placed on an existing pavement.
(4) **Sandwich Pavement.** Overlay pavement sections containing granular separation courses between the old and new impervious surfaces are called sandwich pavements.

b. **Sandwich Pavements.** Regardless of the type of overlay, FAA criteria does not permit the construction of sandwich overlay pavements. They are not allowed because the granular separation course usually becomes saturated with water and provides poor or, at best, unpredictable performance. Saturation of the separation course can be caused by the infiltration of surface water, ingress of ground or capillary water, or the condensation of water from the atmosphere. In any event, the water in the separation course usually cannot beadequatelydrained.Thetrapped water drastically reduces the stability of the overlay.

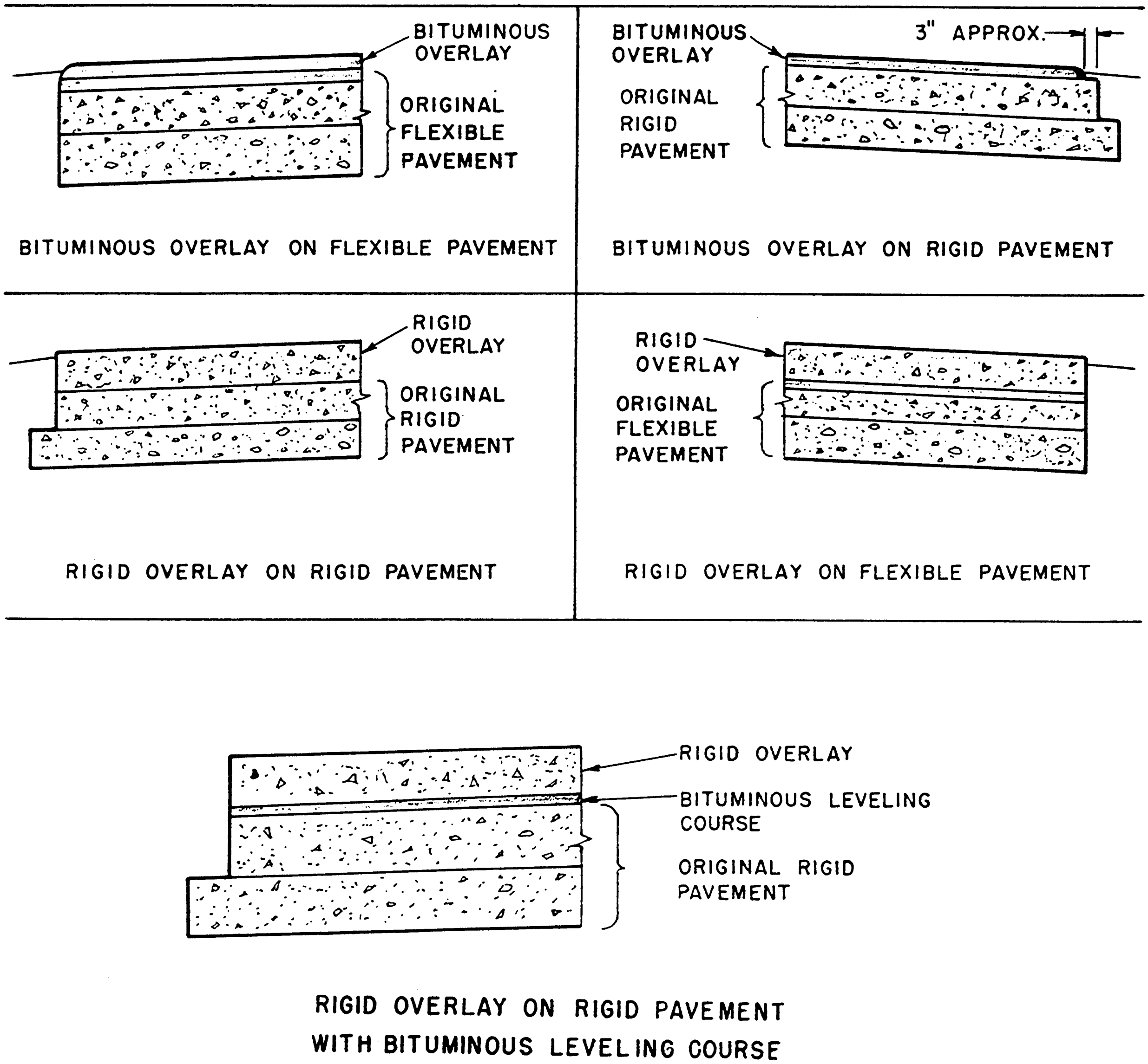


FIGURE 4-1. TYPICAL OVERLAY PAVEMENTS

404. DESIGN OF STRUCTURAL HOT MIX ASPHALT OVERLAYS. Structural hot mix asphalt overlays can be applied to either flexible or rigid pavements. Certain criteria and design assumptions are different for hot mix asphalt overlays of flexible and rigid pavements. The design for procedures are presented separately.

405. HOT MIX ASPHALT OVERLAYS ON EXISTING FLEXIBLE PAVEMENT. The design of structural hot mix asphalt overlays of existing flexible pavements is based on a thickness deficiency approach. That is, the existing pavement is compared to what is needed for a new pavement and any deficiency is made up in the overlay.

a. Calculate New Pavement Requirements. Using the appropriate flexible pavement design curves (Figures 3-2 through 3-15) calculate the thickness requirements for a flexible pavement for the desired load and number of equivalent design departures. A CBR value is required for the subgrade material and subbase. Thicknesses of all pavement layers must be determined.

b. Compare New Pavement Requirements With Existing Pavement. The thickness requirements for a new pavement are compared with the existing pavement to determine the overlay requirements. Adjustments to the various layers of the existing pavement may be necessary to complete the design. This is particularly difficult when overlaying old pavement. Hot mix asphalt surfacing may have to be converted to base, and/or base converted to subbase. Note that a high quality material may be converted to a lower quality material, such as: surfacing to base, or base to subbase. A lesser quality material may not be converted to a higher quality material. For example, excess subbase cannot be converted to base. The equivalency factors shown in Tables 3-6 through 3-8 may be used as guidance in the conversion of layers. It must be recognized that the values shown in Tables 3-6 through 3-8 are for new materials and the assignment of factors for existing pavements must be based on judgment and experience. Surface cracking, high degree of oxidation, evidence of low stability, etc., are a few of the considerations which would tend to reduce the equivalency factor. Any hot mix asphalt layer located between granular courses in the existing pavement should be evaluated inch for inch as granular base or subbase course.

c. Example. To illustrate the procedure of designing a hot mix asphalt overlay, assume an existing taxiway pavement composed of the following section. The subgrade CBR is 7, the hot mix asphalt surface course is 4 inches (100 mm) thick, the base course is 6 inches (150 mm) thick, the subbase is 10 inches (250 mm) thick, and the subbase CBR is 15. Frost action is negligible. Assume the existing pavement is to be strengthened to accommodate a dual wheel aircraft weighing 100,000 pounds (45 000 kg) and an annual departure level of 3,000. The flexible pavement required (referring to Figure 3-3) for these conditions is:

Hot mix asphalt Surface	4 inches (100 mm)
Base	9 inches (230 mm)
Subbase	10 inches (250 mm)
Total pavement thickness	23 inches (585 mm)

The total pavement thickness must be 23 inches (585 mm) in order to protect the CBR 7 subgrade. The combined thicknesses of surfacing and base must be 13 inches (330 mm) to protect the CBR 15 subbase. The existing pavement is 3 inches (75 mm) deficient in total pavement thickness. All of the thickness deficiency is in the base course. For the sake of illustration, assume the existing hot mix asphalt surface is in such a condition that surfacing can be substituted for base at an equivalency ratio of 1.3 to 1. Converting 2.5 inches (64 mm) of surfacing to base yields a base course thickness of 9.2 inches (234 mm) leaving 1.5 inches (40 mm) of unconverted surfacing. A 2.5 inch (64 mm) overlay would be required to achieve a 4 inch (100 mm) thick surface.

d. Summary. Structurally, a 2.5 inch thick overlay should satisfy the design conditions. The overlay thickness calculated from structural considerations should be compared with that required to satisfy geometric requirements. Geometric requirements include, for example, provision of drainage, correcting crown and grade, meeting grade of other adjacent pavements and structures, etc. The most difficult part of designing hot mix asphalt overlays for flexible pavements is the determination of the properties of the existing pavement. Subgrade and subbase CBR values can be determined by conducting field inplace CBR tests. Field CBR tests should be performed in accordance with the procedures given in Manual Series No. 10 (MS-10 by The Asphalt Institute. See Appendix 4. The subgrade and subbase must be at the equilibrium moisture content when field CBR tests are conducted. Normally a pavement which has been in place for at least 3 years will be in equilibrium. Procedures for calculating CBR values from NDT tests are also available. Layer conversions, i.e., converting base to subbase, etc., are largely a matter of engineering judgment. When performing the conversions, it is recommended that any converted thicknesses not be rounded off.

406. HOT MIX ASPHALT OVERLAY ON EXISTING RIGID PAVEMENT. The design of a hot mix asphalt overlay on an existing rigid pavement is also based on a thickness deficiency approach. However, new pavement thickness requirements for rigid pavements are used to compare with the existing rigid pavement. The formula for computing overlay thickness is as follows:

$$t = 2.5(Fh_d - C_b h_e)$$

where:

t = thickness of hot mix asphalt overlay, inches (mm).

F = a factor which controls the degree of cracking in the base rigid pavement.

h_d = thickness of new rigid pavement required for design conditions, inches (mm). Use the exact value for h_d; do not round off. In calculating h_d use the k value of the existing foundation and the flexural strength of the existing concrete as design parameters.

C_b = a condition factor which indicates the structural integrity of the existing rigid pavement. Value ranges from 1.0 to 0.75.

h_e = thickness of existing rigid pavement, inches (mm).

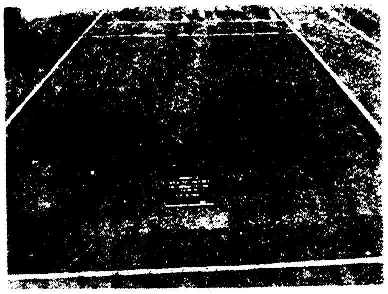
a. F Factor. The "F" factor is an empirical method of controlling the amount of cracking which will occur in the rigid pavement beneath the hot mix asphalt overlay. It is a function of the amount of traffic and the foundation strength. The assumed failure mode for a hot mix asphalt overlay on an existing rigid pavement is that the underlying rigid pavement cracks progressively under traffic until the average size of the slab pieces reaches a critical value. Further traffic beyond this point results in shear failures within the foundation producing a drastic increase in deflections. Since high strength foundations can better resist deflection and shear failure, the F factor is a function of subgrade strength as well as traffic volume. Photographs of various overlay and base pavements shown in Figure 4-2 illustrate the meaning of the "F" factor. Figures 4-2 a, b, and c show how the overlay and base pavements fail as more traffic is applied to a hot mix asphalt overlay on an existing rigid pavement. Normally an F factor of 1.00 is recommended unless the existing pavement is in quite good condition, see paragraph 406b.(1) below. Figure 4-3 is a graph which should be used to determine the appropriate F factor for pavements in good condition.

b. C_b Factor. The condition factor "C_b" applies to the existing rigid pavement. The "C_b" factor is an assessment of the structural integrity of the existing pavement.

(1) Selection of C_b Factor. The overlay formula is rather sensitive to the "C_b" value. A great deal of care and judgment are necessary to establish the appropriate "C_b". NDT can be a valuable tool in determining an proper value. A "C_b" value of 1.0 should be used when the existing slabs contain nominal structural cracking and 0.75 when the slabs contain structural cracking. The designer is cautioned that the range of "C_b" values used in hot mix asphalt overlay designs is different from the "C_r" values used in rigid overlay pavement design. A comparison of "C_b" and "C_r" and the recommended "F" factor to be used for design is shown below:

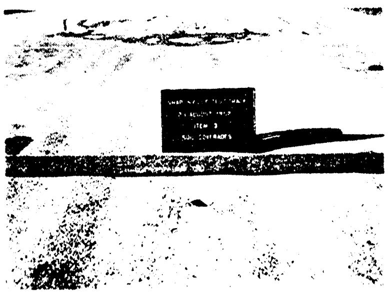
C_b	C_r	Recommended F factor
0.35 to 0.50	0.75 to 0.80	1.00
0.51 to 0.75	0.81 to 0.90	1.00
0.76 to 0.85	0.91 to 0.95	1.00
0.86 to 1.00	0.96 to 1.00	use Figure 4-3

The minimum "C_b" value is 0.75. A single "C_b" should be established for an entire area. The "C_b" value should not be varied along a pavement feature. Figures 4-4 and 4-5 illustrate "C_b" values of 1.0 and 0.75, respectively.



SURFACE OF OVERLAY

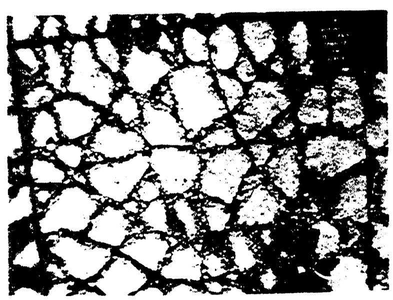
BASE PAVEMENT



SURFACE OF OVERLAY

BASE PAVEMENT

SURFACE OF OVERLAY



BASE PAVEMENT

FIGURE 4-2. ILLUSTRATION OF VARIOUS "F" FACTORS FOR HOT MIX ASPHALT OVERLAY

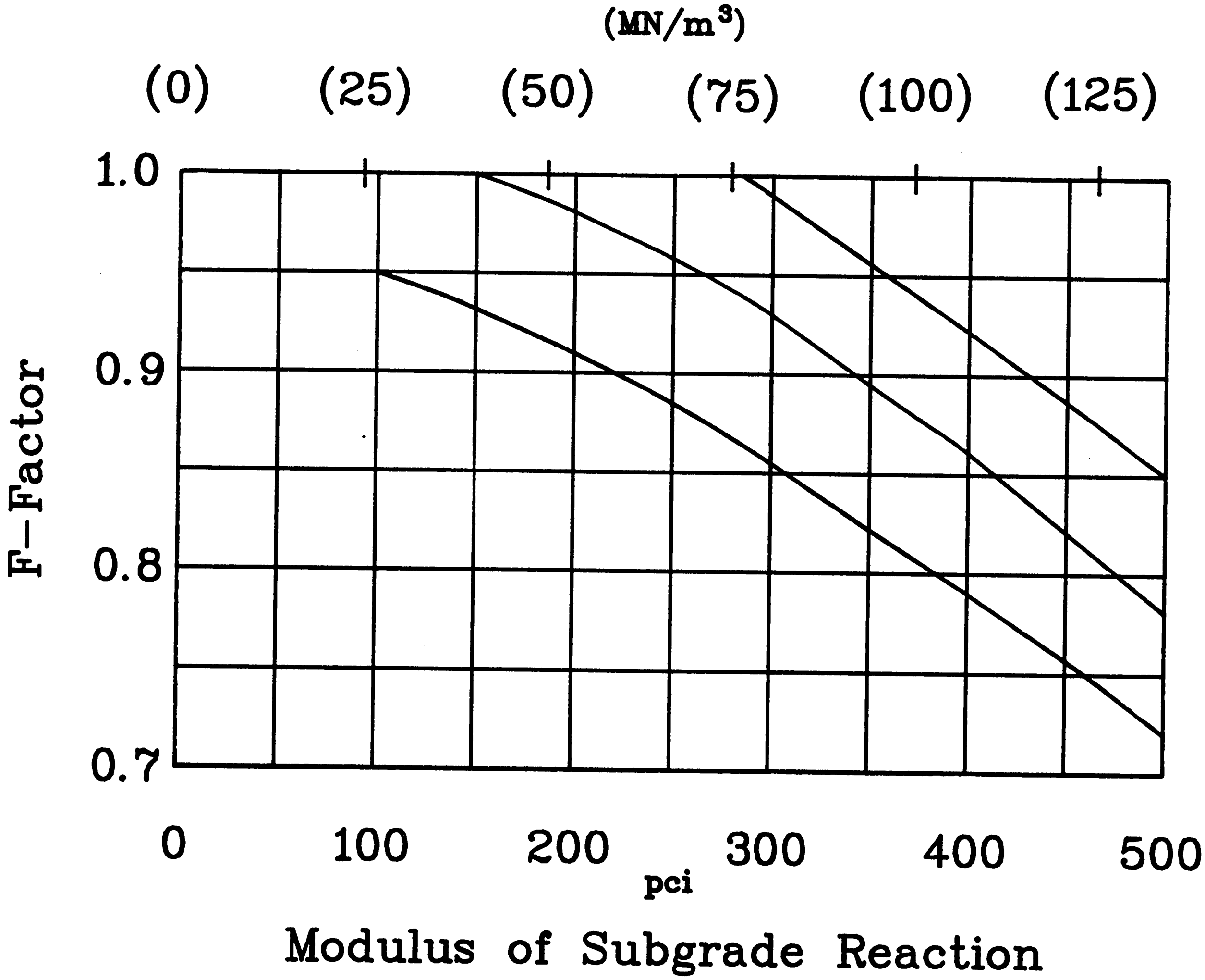
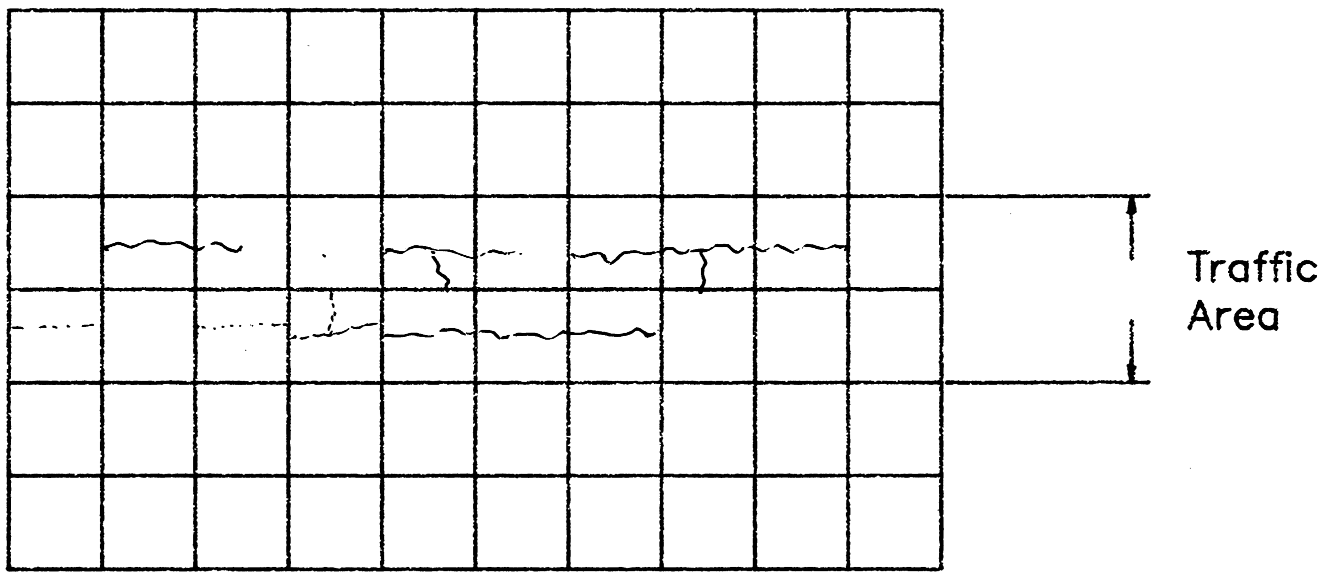


FIGURE 4.3 GRAPH OF "F" FACTOR VS. MODULUS OF SUBGRADE REACTION FOR DIFFERENT TRAFFIC LEVELS



Legend:
Crack Width
- - - - Less Than 1/4 inch (6mm)
——— Greater Than 1/4 inch (6mm)

Note:
50% of Slabs Within Traffic Area Broken Into 2 to 3 Pieces. No Working Cracks, Corner Breaks, or Faulted Joints.

FIGURE 4.4 ILLUSTRATION OF A "C_b" FACTOR OF 1.0 FOR HOT MIX ASPHALT OVERLAY DESIGN

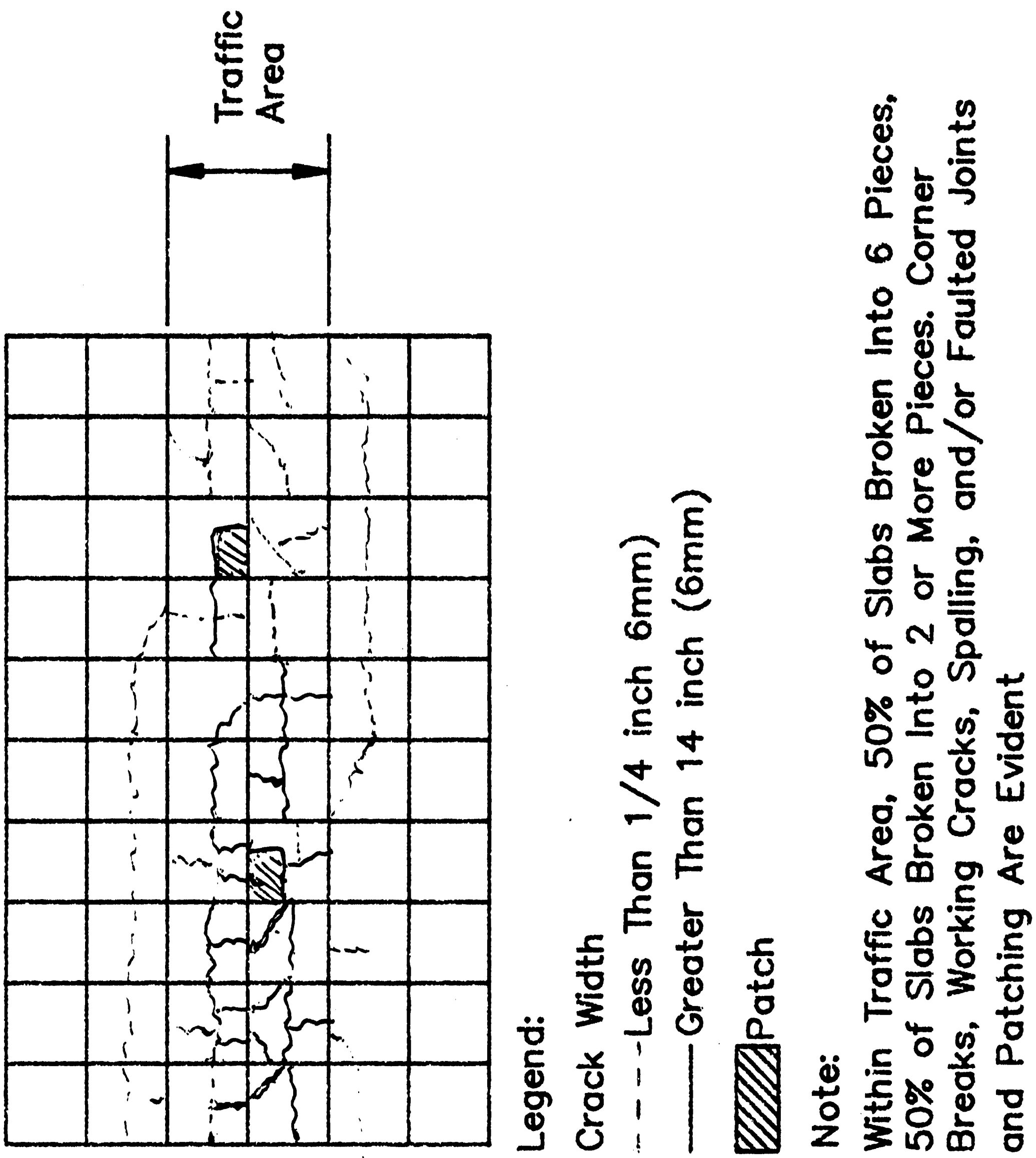


FIGURE 4-5. ILLUSTRATION OF A "C_b" FACTOR OF 0.75 FOR HOT MIX ASPHALT OVERLAY DESIGN

(2) **Increasing C_b Factor.** A value of C_b lower than 0.75 represents a severely cracked base slab, which would not be advisable to overlay without modification due to the likelihood of severe reflection cracking. See paragraph 406 f. In some instances it may be advantageous to replace several slabs and restore load transfer along inadequate joints to raise the C_b value. Increasing the C_b value will decrease the required overlay thickness. A detailed condition survey of the existing pavement which examines the subsurface drainage conditions, structural capacity of the slabs, foundation strength, flexural strength of the concrete, load transfer along joints and thickness of the component layers is strongly encouraged to properly design a hot mix asphalt overlay.

c. **Example.** An example of the hot mix asphalt overlay design method is given below:

(1) **Assumptions.** Assume an existing rigid pavement 12 inches (305 mm) thick is to be strengthened to accommodate 3000 departures of a dual wheel aircraft weighing 180,000 pounds (81,800 kg). The flexural strength of the existing concrete is 725 psi (5.00 MN/m^2) and the foundation modulus is 300 pci (81.6 MN/m^3). The condition factor of the existing pavement is 0.95.

(2) **Single Slab Thickness.** Compute the single slab thickness required to satisfy the design conditions given in (1) above. Using Figure 3-17 the slab thickness is found to be 13.9 inches (353 mm). The F factor is determined from Figure 4-3 and equals 0.93. Applying the overlay formula given in paragraph 406 yields:

$$t = 2.5\,(0.93 \times 13.9 - 0.95 \times 12)$$
$$t = 3.82 \text{ inches} (97 \text{ mm})$$

This thickness would be rounded up to 4 inches (100 mm) for practicality of construction.

d. **Previously Overlaid Rigid Pavement.** The design of a hot mix asphalt overlay for a rigid pavement which already has an existing hot mix asphalt overlay is slightly different. The designer should treat the problem as if the existing hot mix asphalt overlay were not present, calculate the overlay thickness required, and then adjust the calculated thickness to compensate for the existing overlay. If this procedure is not used, inconsistent results will often be produced.

e. **Example.** An example of a hot mix asphalt overlay design for a rigid pavement which already has an existing hot mix asphalt overlay is given below:

(1) **Assumptions.** An example of the procedure follows. Assume an existing pavement consists of a 10-inch (255 mm) rigid pavement with a 3-inch (75 mm) hot mix asphalt overlay. The existing pavement is to be strengthened to be equivalent to a single rigid pavement thickness of 14 inches (355 mm). Assume an "F" factor of 0.9 and "C_b" of 0.9 are appropriate for the existing conditions.

(2) **Ignore Existing Overlay.** Calculate the required thickness of hot mix asphalt overlay as if the existing 3-inch (75 mm) overlay were not present.

$$t = 2.5\,(0.9 \times 14 - 0.9 \times 10)$$
$$t = 9 \text{ inches} (230 \text{ mm})$$

(3) **Thickness Allowance For Existing Overlay.** An allowance is then made for the existing hot mix asphalt overlay. In this example assume the existing overlay is in such a condition that its effective thickness is only 2.5 inches (64 mm). The required overlay thickness would then be 9 - 2.5 = 6.5 inches (165 mm). The determination of the effective thickness of the existing overlay is a matter of engineering judgment.

e. **Limitations.** The formula for hot mix asphalt overlay thickness assumes the existing rigid pavement will support load through flexural action. As the overlay thickness becomes greater, at some point the existing rigid pavement will tend to act more like a high quality base material. When this condition is reached, the overlay should be designed as a flexible pavement with the existing pavement treated as a high quality base course.

f. **Crack and Seat.** If the condition of the existing rigid pavement is very poor, i.e., extensive structural cracking, joint faulting, "D" cracking etc. consideration should be given to using the "crack and seat" technique. The crack and seat technique involves purposely breaking the existing rigid pavement and then rolling the broken pieces to firmly seat them in the foundation. A hot mix asphalt layer is then placed over the pavement. This type of section is designed as a flexible pavement treating the broken rigid pavement as base course. The severity of reflection cracking is often reduced with this type of construction. On the other hand, any life remaining in the existing rigid pavement is essentially destroyed.

407. NONSTRUCTURAL HOT MIX ASPHALT OVERLAYS. In some instances overlays are required to correct nonstructural problems such as restoration of crown, improve rideability, etc. Thickness calculations are not required in these situations, as thickness is controlled by other design considerations or minimum practical overlay thickness. Information concerning runway roughness correction can be found in FAA Report No. FAA-RD-75-110, Methodology for Determining, Isolating and Correcting Runway Roughness. See Appendix 4.

408. REFLECTION CRACKING IN HOT MIX ASPHALT OVERLAYS. Reflection cracking is often a problem in hot mix asphalt overlays particularly overlays of rigid pavement. Numerous materials and techniques have been tried attempting to solve the problem with varying degrees of success. The following methods have met with some success:

a. Coarse Aggregate Binders. The use of coarse aggregate binder course is recommended where economically feasible. Use of the largest practical size coarse aggregate in the hot mix asphalt layer immediately above the existing pavement is recommended. This practice provides some measure of protection against reflection cracking.

b. Engineering Fabrics. Recent research studies have shown that nonwoven fabric membranes are effective in retarding reflection cracking. See DOT/FAA/PM-84/9, I, Appendix 4. While fabrics probably will not eliminate reflection cracking all together, they do provide some degree of water-proofing beneath reflection cracks thus protecting the existing pavement and foundation. At present, the water-proofing capability of fabrics, assuming the capacity of the asphalt impregnated fabric to resist rupture is not lost, appears to be the most significant contribution fabrics provide in a hot mix asphalt overlay system. Existing pavements, whether flexible or rigid, that show evidence of excessive deflections, substantial thermal stresses, and/or poor drainage, probably will exhibit no improvement by including a fabric in a structural overlay. The following conditions are recommended for fabric usage:

(1) Fabric Properties. The fabric should have a tensile strength of at least 90 lbs (41 kg) when tested in accordance with ASTM D 1682 and a density in the range of 3 to 5.5 ozs per square yard (70 to 130 grams per square meter).

(2) Application. Fabric membranes should not be used where the horizontal displacements exceed 0.05 inch (1.3 mm) or where vertical displacements will exceed 0.02 inch (0.5 mm). Fabric should not be used when the overlay thickness is less than 3 inches (75 mm) or more than 7 inches (178 mm). To date only nonwoven fabrics have been studied in the above referenced research effort. It is anticipated that woven fabrics may also be used if the above conditions are satisfied.

(3) Tack Coat. The proper amount of tack coat applied to the fabric is critical. An emulsified asphalt applied at a rate of from 0.15 to 0.30 gallons per square yard (0.7 to 1.4 liters per square meter) is recommended. The optimum amount of tack coat will depend on the type of fabric and the surface on which the fabric is placed.

409. DESIGN OF CONCRETE OVERLAYS. Concrete overlays can be constructed on existing rigid or flexible pavements. The minimum allowable thickness for concrete overlays is 5 inches (127 mm) when placed on a flexible pavement, directly on a rigid pavement, or on a leveling course. The minimum thickness of concrete overlay which is bonded to an existing rigid pavement is 3 inches (75 mm). The design of concrete overlays is based on a thickness deficiency approach. The existing base pavement and overlay section are equated to a single slab thickness. The empirical formulas presented were developed from research on test track pavements and observations of in-service pavements.

410. CONCRETE OVERLAY ON FLEXIBLE PAVEMENT. The design of concrete overlays on existing flexible pavements assumes the existing flexible pavement is a foundation for the overlay slab. Overlay slab thickness is based on the design curves in Figures 3-17 through 3-40. The existing flexible pavement should be assigned a k value using Figure 2-5 or 3-16 or by conducting a plate bearing test on the existing flexible pavement or by NDT testing. In any case the k value assigned should not exceed 500. When frost conditions require additional thickness, the use of nonstabilized material below the rigid pavement overlay is not allowed as this would result in a sandwich pavement. Frost protection must be provided by stabilized material.

411. CONCRETE OVERLAY ON RIGID PAVEMENT. The design of concrete overlays on existing rigid pavements is also predicated on the rigid pavement design curves, Figures 3-17 through 3-40. The rigid pavement design curves indicate the thickness of concrete required to satisfy the design conditions for a single thickness of concrete pavement. Use of this method requires the designer to assign a k value to the existing foundation. The k value may be determined by field NDT tests or by bearing tests conducted in test pits cut through the existing rigid pavement, or may be estimated from construction records for the existing pavement. The design of a concrete overlay on a rigid pavement requires an assessment of the structural integrity of the existing rigid pavement. The condition factor should be selected after an extensive pavement condition survey. The selection of a condition factor is a matter of engineering judgment. The use of nondestructive testing (NDT) can be of considerable value in assessing the condition of an existing pavement. NDT can also be used to determine sites for test pits. NDT procedures are given in Advisory Circular 150/5370-11, Use of Nondestructive Testing Devices in the Evaluation of Airport Pavements. See Appendix 4. In order to provide a more uniform assessment of condition factors, the following values are defined:

C_r = 1.0 for existing pavement in good condition -- some minor cracking evident, but no structural defects.

C_r = 0.75 for existing pavement containing initial corner cracks due to loading but no progressive cracking or joint faulting.

C_r = 0.35 for existing pavement in poor structural condition, badly cracked or crushed and faulted joints.

The three conditions discussed above are used to illustrate the condition factor rather than establish the only values available to the designer. Conditions at a particular location may require the use of an intermediate value of C_r within the recommended range. Sketches of three different values of C_r are shown in Figures 4-6, 4-7, and 4-8.

a. Concrete Overlay Without Leveling Course. The thickness of the concrete overlay slab applied directly over the existing rigid pavement is computed by the following formula.

$$h_c = \sqrt[1.4]{h^{1.4} - C_r h_e^{1.4}}$$

where:

h_c = required thickness of concrete overlay
h = required single slab thickness determined from design curves
h_e = thickness of existing rigid pavement
C_r = condition factor

Due to the inconvenient exponents in the above formula, graphic solutions are given in Figures 4-9 and 4-10. These graphs were prepared for only two different condition factors, C_r = 1.0 and 0.75. The use of a concrete overlay pavement directly on an existing rigid pavement with a condition factor of less than 0.75 is not recommended because of the likelihood of reflection cracking. The above equation assumes the flexural strength of the concrete used for the overlay will be approximately equal to that of the base pavement. If the flexural strengths differ by more than 100 psi (0.7 MN/m^2), the following modified equation should be used to determine the required thickness of the overlap

$$h_c = \sqrt[1.4]{1.4h - C_r h_b^{1.4h \times h_e}}$$

where:

h_b = required single slab thickness determined from design curves based on the flexural strength of the base pavement.

Other factors are the same as previous formula.

b. Concrete Overlay With Leveling Course. In some instances it may be necessary to apply a leveling course of hot mix asphalt concrete to an existing rigid pavement prior to the application of the concrete overlay. Under these conditions a different formula for the computation of the overlay thickness is required. When the existing pavement and overlay pavement are separated, the slabs act more independently than when the slabs are in contact with each other. The formula for the thickness of an overlay slab when a leveling course is used is as follows:

$$h_c = \sqrt{h^2 - C_r h_e^2}$$

where: h_c = required thickness of concrete overlay
h = required single slab thickness determined from design curves
h_e = thickness of existing rigid pavement
C_r = condition factor

When the flexural strength of the overlay and of the existing pavements differ by more than 100 psi (0.7 MN/m^2), the equation is modified as follows:

$$h_c = \sqrt{2h - 2C_r \frac{h}{h_b} h_e}$$

where: h_b = required single slab thickness determined from design curves based on the flexural strength of the base pavement.

The leveling course must be constructed of highly stable hot mix asphalt concrete. A granular separation course is not allowed as this would constitute sandwich construction. Graphic solutions of the above equation are shown in Figures 4-11 and 4-12. These graphs were prepared for condition factors of 0.75 and 0.35. Other condition actors between these values can normally be computed to sufficient accuracy by interpolation.

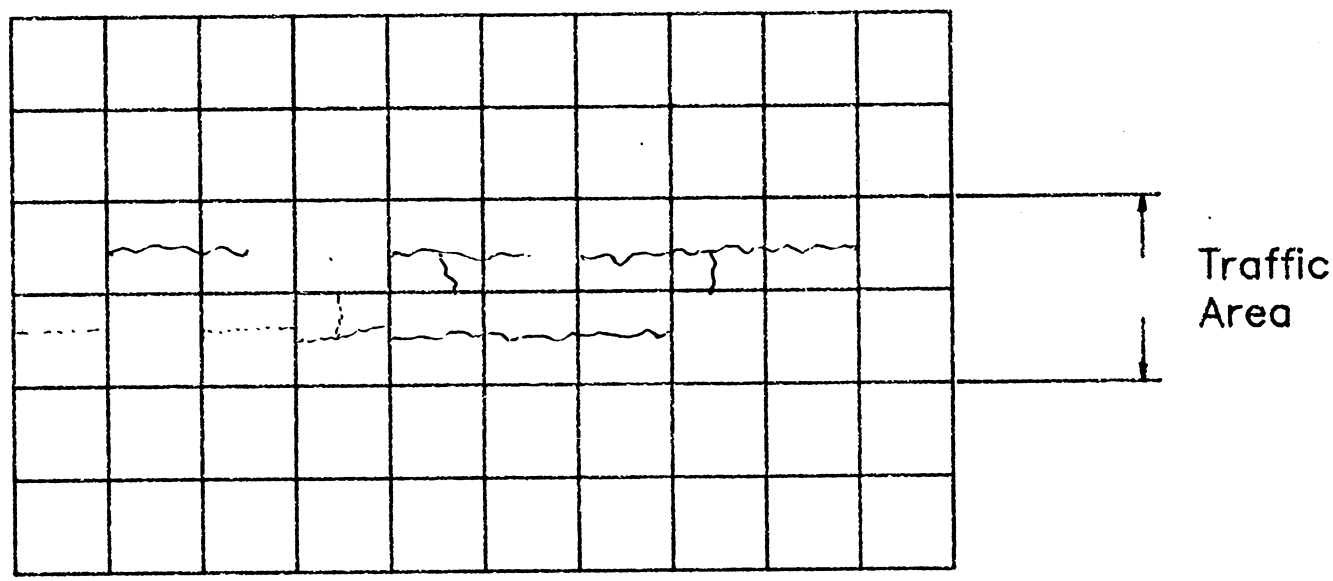
412. BONDED CONCRETE OVERLAYS. Concrete overlays bonded to existing rigid pavements are sometimes used under certain conditions. By bonding the concrete overlay to the existing rigid pavement, the new section behaves as monolithic slab. The thickness of bonded overlay required is computed by subtracting the thickness of the existing pavement from the thickness of the required slab thickness determined from design curves.

$$h_c = h - h_e$$

where: h_c = required thickness of concrete overlay.
h = required single slab thickness determined from design curves using the flexural strength of the existing concrete.
h_e = thickness of existing rigid pavement.

Bonded overlays should be used only when the existing rigid pavement is in good condition. Defects in the existing pavement are more likely to reflect through a bonded overlay than other types of concrete overlays. The major problem likely to be encountered with bonded concrete overlays is achieving adequate bond. Elaborate surface preparation and exacting construction techniques are required to insure bond.

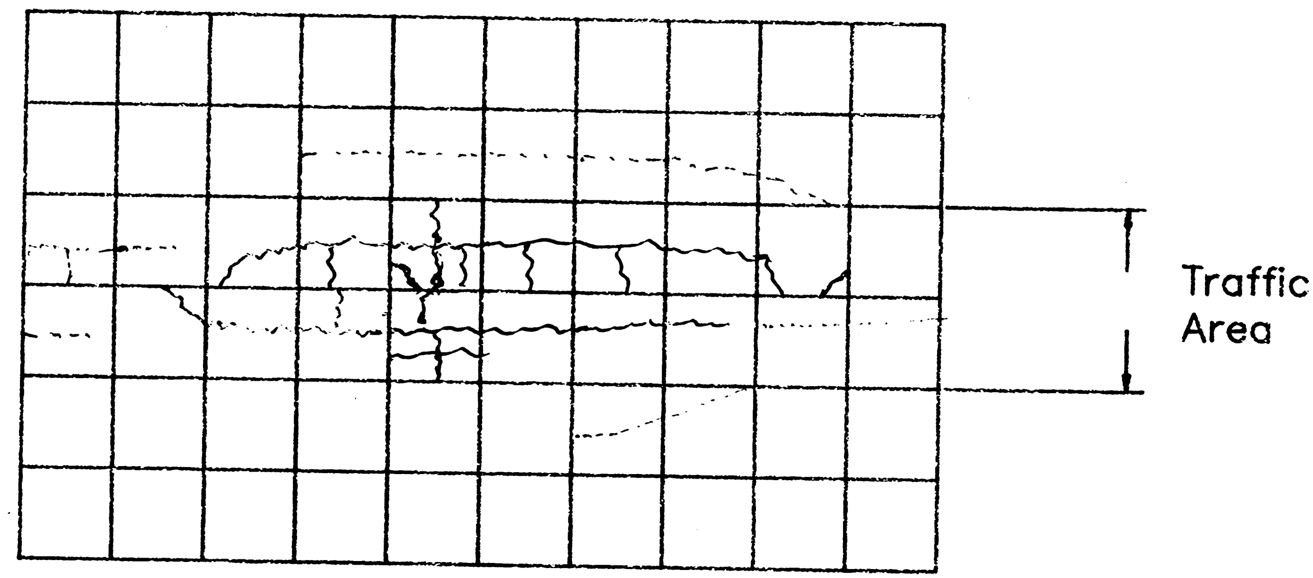
413. JOINTING OF CONCRETE OVERLAYS. Where a rigid pavement is to receive the overlay, some modification to jointing criteria may be necessary because of the design and joint arrangement of the existing pavement. The following points may be used as guides in connection with the design and layout of joints in concrete overlays.



Legend:
Crack Width
- - - - Less Than 1/4 inch (6mm)
——— Greater Than 1/4 inch (6mm)

Note:
50% of Slabs Within Traffic Area Broken Into 2 to 3 Pieces. No Working Cracks, Corner Breaks, or Faulted Joints.

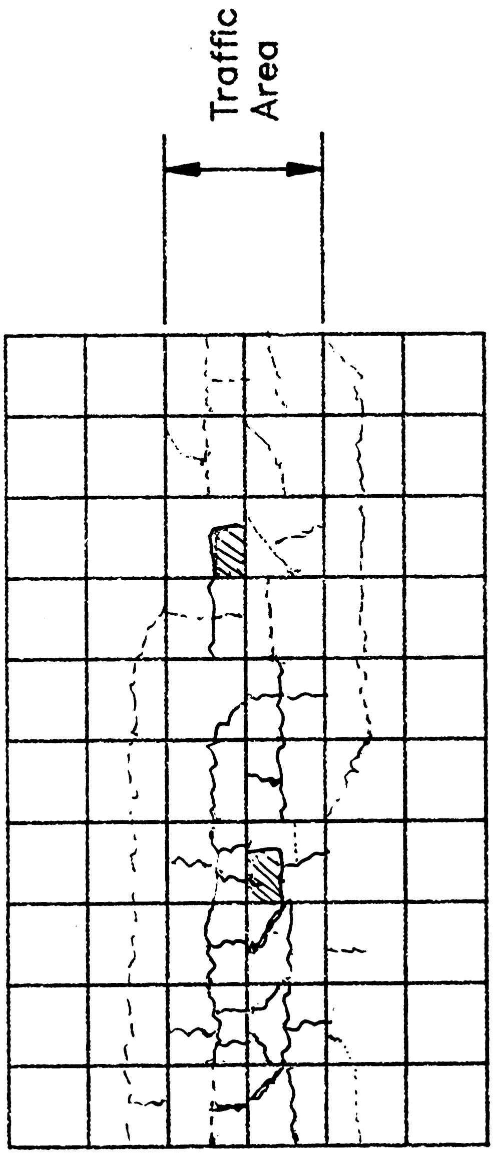
FIGURE 4-6. ILLUSTRATION OF A "Cr" FACTOR OR 1.0



Legend:
Crack Width
- - - - Less Than 1/4 inch (6mm)
——— Greater Than 1/4 inch (6mm)

Note:
Within Traffic Area, 10% of Slabs Broken Into 6 Pieces, 75% of Slabs Broken Into 2 or More Pieces. Some Corner Breaks, Working Cracks, and Spalling Are Evident

FIGURE 4-7. ILLUSTRATION OF A "C" FACTOR OF 0.6



Legend:

Crack Width

- - - -Less Than 1/4 inch 6mm)

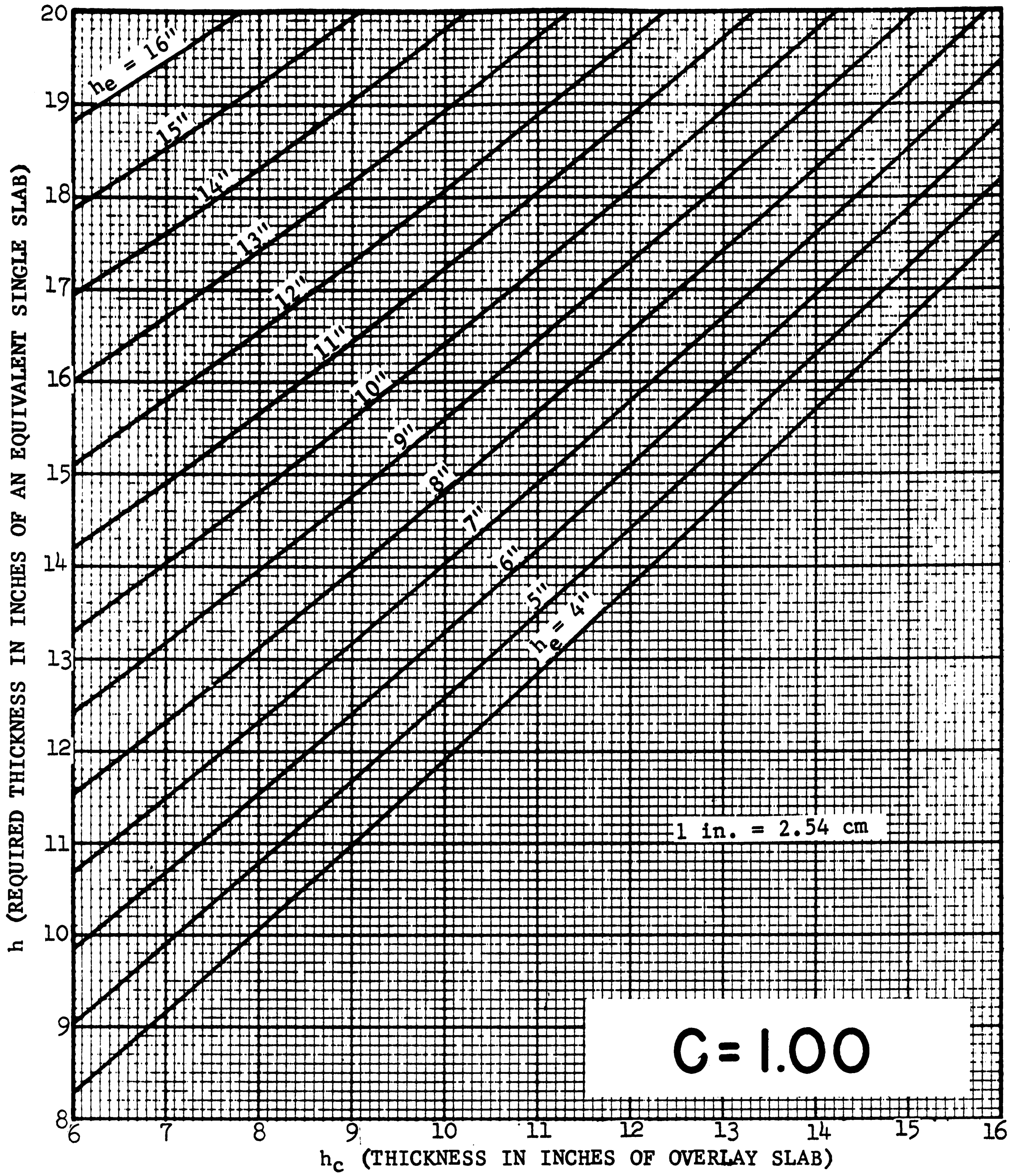
——Greater Than 14 inch (6mm)

Patch

Note:

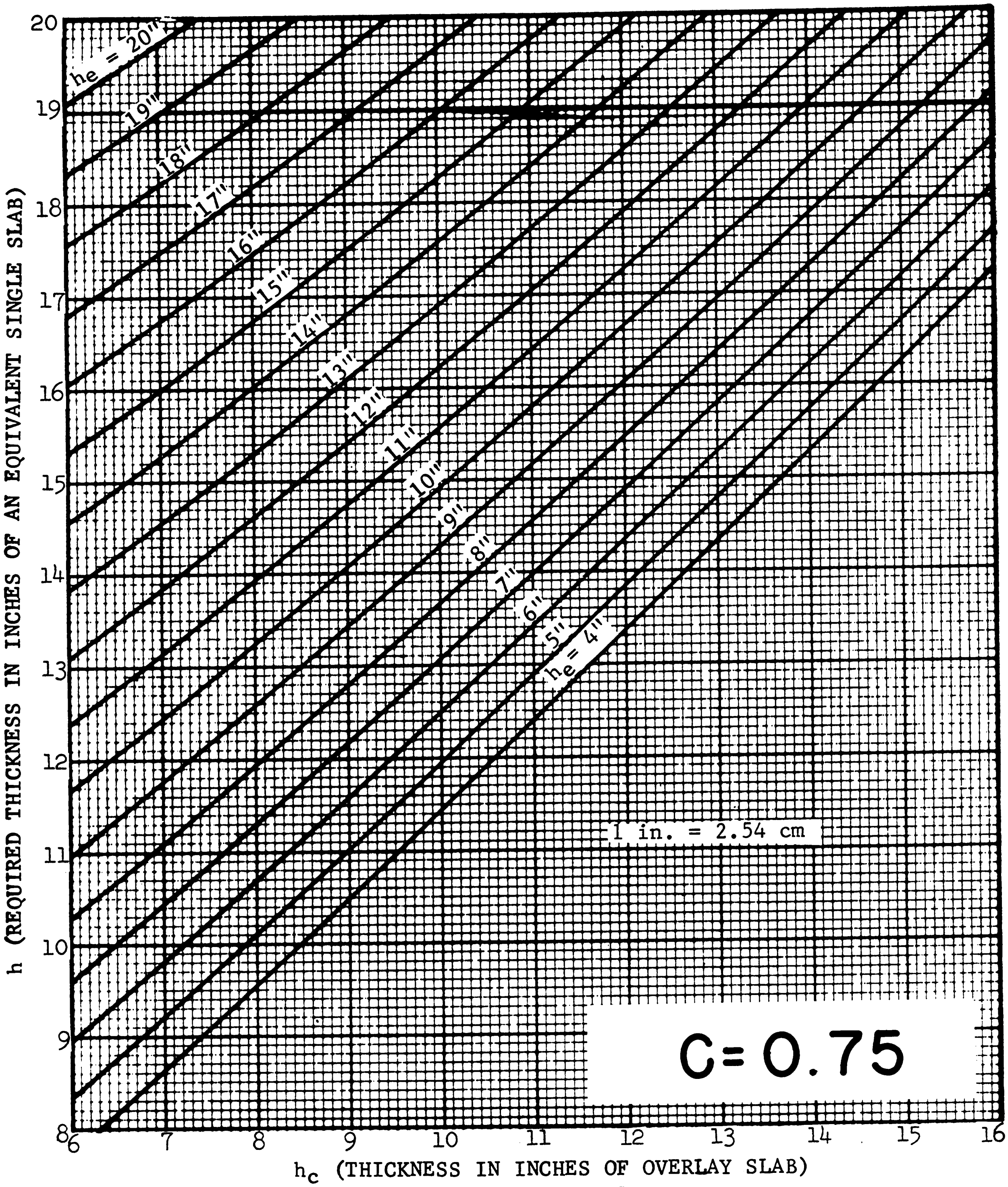
Within Traffic Area, 50% of Slabs Broken Into 6 Pieces, 50% of Slabs Broken Into 2 or More Pieces. Corner Breaks, Working Cracks, Spalling, and/or Faulted Joints and Patching Are Evident

FIGURE 4-8. ILLUSTRATION OF A "C" FACTOR OF 0.35



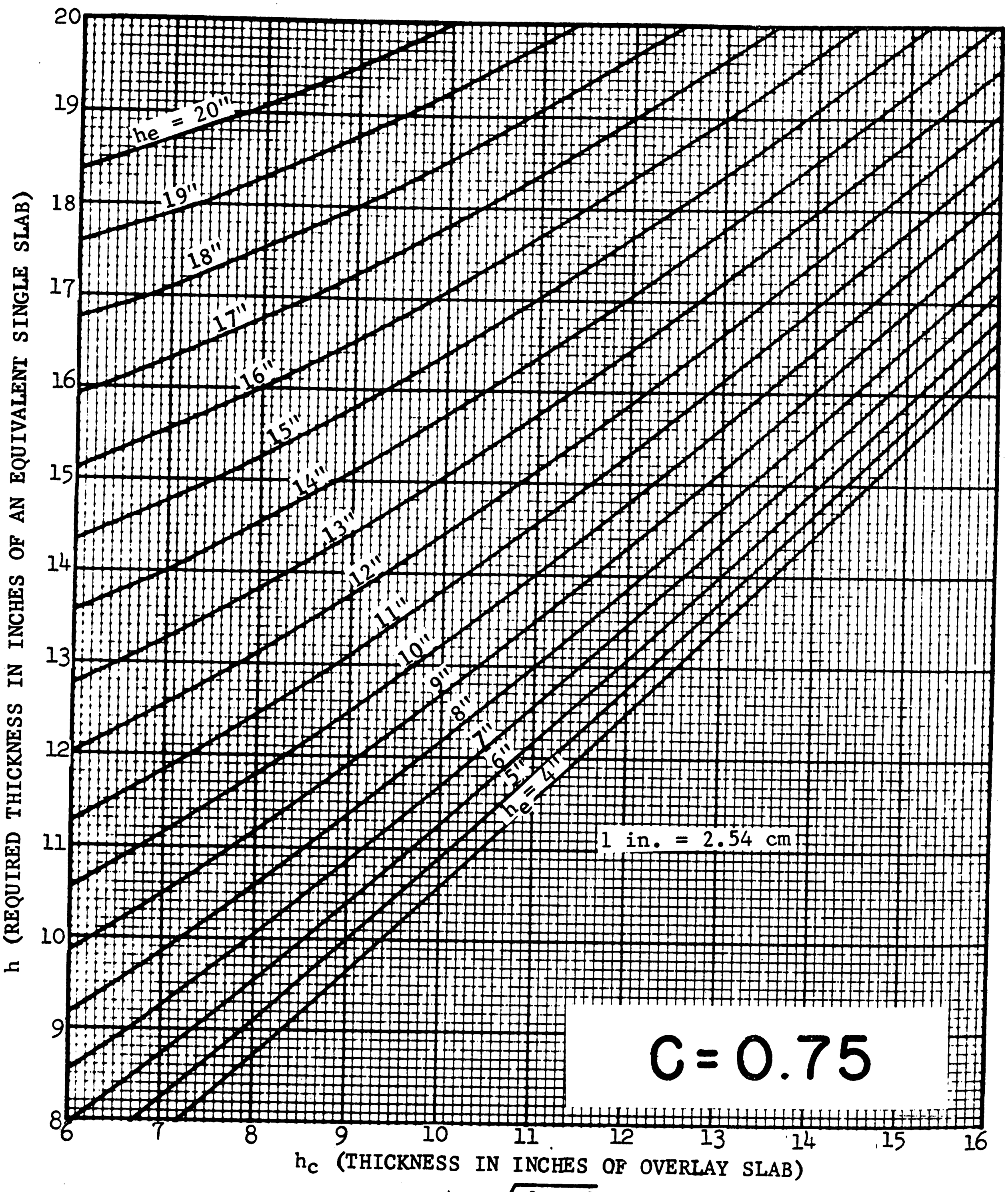
$$h_c = \sqrt[1.4]{h_1^{1.4} - Ch_e^{1.4}}$$

FIGURE 4-9. CONCRETE OVERLAY ON RIGID PAVEMENT



$$h_c = \sqrt[1.4]{h^{1.4} - Ch_e^{1.4}}$$

FIGURE 4-10. CONCRETE OVERLAY ON RIGID PAVEMENT



$$h_c = \sqrt{h^2 - Ch_e^2}$$

FIGURE 4-11. CONCRETE OVERLAY ON RIGID PAVEMENT

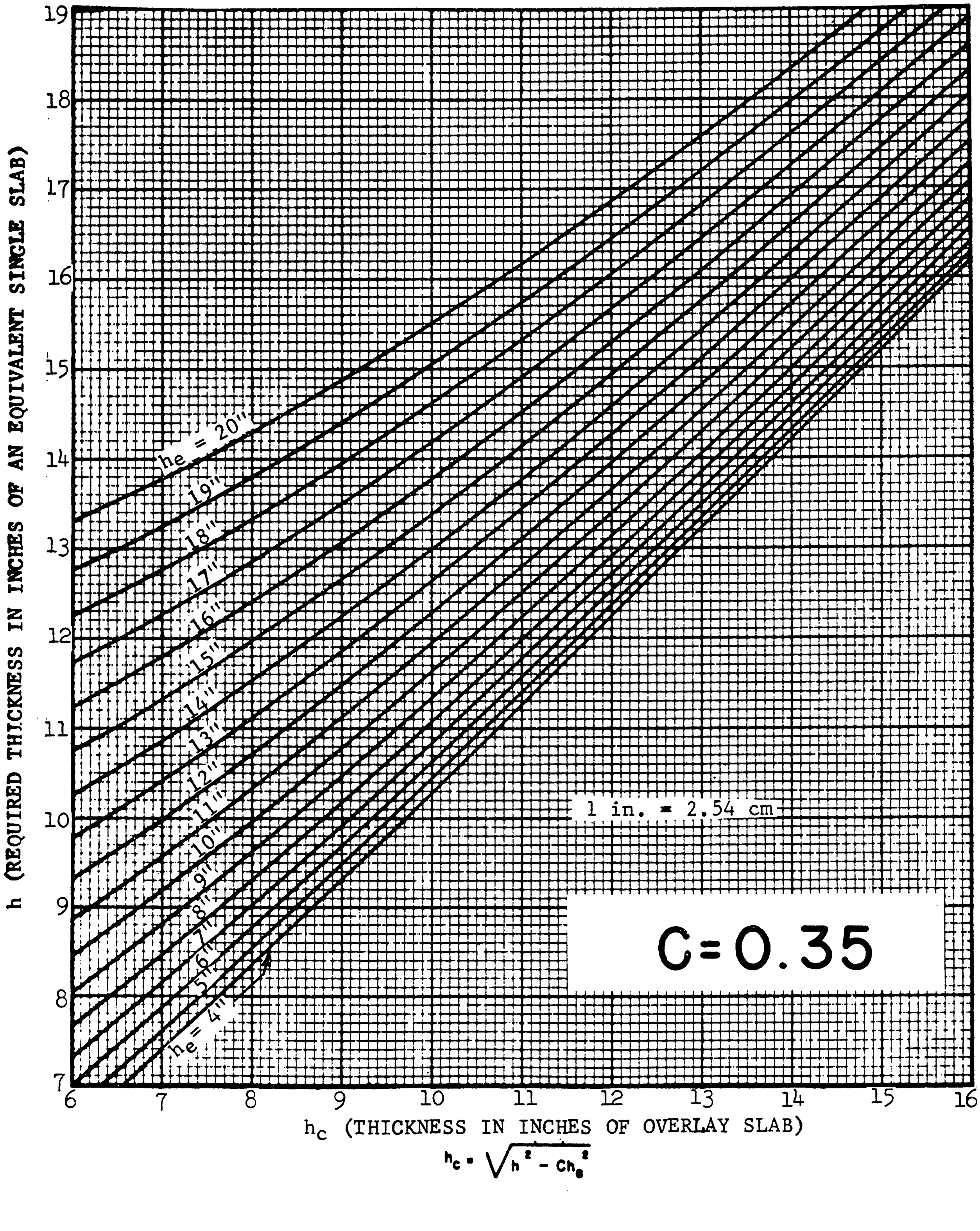


FIGURE 4-12. CONCRETE OVERLAY ON RIGID PAVEMENT

a. Joint Types. Joints need not be of the same type as in the old pavement except for some bonded overlay applications.

b. Expansion Joints. It is not necessary to provide an expansion joint for each expansion joint in the old pavement; however, a saw cut or plane of weakness should be provided within 1 foot (0.3 m) of the existing expansion joint.

c. Timing. The timing for sawing joints is extremely critical on concrete overlays.

d. Contraction Joints. Contraction joints in partially bonded or unbonded overlays may be placed directly over or within 1 foot (0.3 m) of existing expansion, construction, or contraction joints. Joints in bonded overlays should be located within 1/2 inch (12 mm) of joints in the existing base pavement. Should spacing result in slabs too long to control cracking, additional intermediate contraction joints may be necessary.

e. Joint Pattern. If a concrete overlay with a leveling course is used, the joint pattern in the overlay does not have to match the joint pattern in the existing pavement.

f. Reinforcement. Overlay slabs longer or wider than 20 feet (6.1 m) should be reinforced regardless of overlay thickness.

414. PREPARATION OF THE EXISTING SURFACE FOR THE OVERLAY. Before proceeding with construction of the overlay, steps should be taken to correct all defective areas in the existing surface, base, subbase, and subgrade. Careful execution of this part of an overlay project is essential as a poorly prepared base pavement will result in an unsatisfactory overlay. Deficiencies in the base pavement will often be reflected in the overlay.

a. Existing Flexible Pavements. Failures in flexible pavements may consist of pavement breakups, potholes and surface irregularities, and depressions.

(1) Removal And Replacement. Localized areas of broken pavement will have to be removed and replaced with new pavement. This type of failure is usually encountered where the pavement is deficient in thickness, the subgrade consists of unstable material, or poor drainage has reduced the supporting power of the subgrade. To correct this condition, the subgrade material should be replaced with a select subgrade soil or by installation of proper drainage facilities; this is the first operation to be undertaken in repairing this type of failure. Following the correction of the subgrade condition, the subbase, base, and surface courses of the required thickness should be placed. Each layer comprising the total repair should be thoroughly compacted before the next layer is placed.

(2) Irregularities And Depressions. Surface irregularities and depressions, such as shoving, rutting, scattered areas of settlement, and occasional "birdbaths" should be leveled by rolling, where practical, or by filling with suitable hot mix asphalt mixtures. If the "birdbaths" and settlements are found to exist over extensive areas, a hot mix asphalt leveling course may be required as part of the overlay. The leveling course should consist of a high-quality hot mix asphalt concrete. Scattered areas requiring leveling or patching may be repaired with hot mix asphalt patch mixtures.

(3) Bleeding Surface. A bleeding surface may detrimentally affect the stability of the overlay and for this reason any excess hot mix asphalt material accumulated on the surface should be bladed off if possible. In some instances, a light application of fine aggregates may blot up the excess material, or a combination of the two processes may be necessary.

(4) Cracks And Joints. For cracks, and joints, 3/8 inch (10 mm) or more in width, old joint and crack filler should be removed and, if vegetation is present, a sterilant applied. The cracks and joints should then be filled with a lean mixture of sand and liquid bituminous material. This mixture should be well tamped in place, leveled with the pavement surface and any excess removed. The material should be allowed to dry to a hardened condition prior to overlay placement.

(5) **Potholes.** Potholes should be thoroughly cleaned and filled with a suitable bituminous mixture and tamped in place.

b. **Existing Rigid Pavements.** In rigid pavements, narrow transverse, longitudinal, and corner cracks will need no special attention unless there is an appreciable amount of displacement and faulting between the separate slabs. If the subgrade is stable and no pumping has occurred, the low areas can be taken care of as part of the overlay and no other corrective measures are needed. On the other hand, if pumping has occurred at the slab ends or the slabs are subject to rocking under the movement of aircraft, subgrade support should be improved by pumping cement grout or asphalt cement under the pavement to fill the voids that have developed. Pressure grouting requires considerable skill to avoid cracking slabs or providing uneven support for the overlay.

(1) **Slab Removal And Replacement.** If the pavement slabs are badly broken and subject to rocking because of uneven bearing on the subgrade, the rocking slabs can be broken into smaller slabs to obtain a more firm seating. Badly broken slabs that do not rock will not require repairs since the criteria make adjustments for such a condition in the pavement thickness. In some cases, it may be desirable to replace certain badly broken slabs with new slabs before starting construction of the overlay. The decision in such cases will have to be made according to the merits of the individual project.

(2) **Leveling Course.** Where the existing pavement is rough due to slab distortion, faulting, or settlement, a provision should be made for a leveling course of hot mix asphalt concrete before the overlay is commenced.

(3) **Cracks And Joints.** Cracks, and joints, 3/8 inch (10 mm) or more in width, should be filled with a lean mixture of sand and liquid bituminous material. This mixture should be tamped firmly in place, leveled with the pavement surface and any excess removed.

(4) **Surface Cleaning.** After all repairs have been completed and prior to the placing of the overlay, the surface should be swept clean of all dirt, dust, and foreign material that may tend to break the bond between the overlay and the existing pavement. Any extruding joint-sealing material should be trimmed from rigid pavements.

(5) **Bonded Concrete Overlays.** Bonded concrete overlays will require special attention to insure bond with the existing pavement. Surface cleaning and preparation by shot peening or mechanical texturing by cold milling are two techniques which have been used to provide a surface which will allow bonding. Adequate bond has been achieved by placing the overlay directly on the dry prepared surface. In other instances, bond was achieved by placing a neat cement grout on the prepared surface immediately ahead of the overlay placement.

415. MATERIALS AND METHODS. With regard to quality of materials and mixes, control tests, methods of construction, and workmanship the overlay pavement components are governed by AC 150/5370-10, Standards for Specifying Construction of Airports.

a. **Tack Coat.** If a hot mix asphalt overlay is specified, the existing pavement should receive a light tack coat (Item P-603) or fog coat immediately after cleaning. The overlay should not extend to the edges of the pavement but should be cut off approximately 3 inches (75 mm) from each edge.

b. **Forms.** Should the existing pavement require drilling to provide anchorage for the overlay pavement forms, the size and number of holes should be the minimum necessary to accomplish that purpose. Holes should not be located close to joints or cracks. Location of holes for form anchors should be such as to avoid causing additional cracking or spalling.

416. NEW OVERLAY MATERIALS. In recent years, some new pavement overlay materials have been used with varying degrees of success. These materials include fibrous concrete, roller compacted concrete, and rubberized asphalt. Use of materials other than conventional portland cement concrete (Item P-501) or Plant Mix Bituminous Surface (Item P-401) require special approval on a case-by-case basis.

417. **POSSIBLE ANOMALIES.** The basic design concepts applied to rigid and flexible pavements are different because of differences in behavior and in failure mechanisms for these pavements. These differences can produce anomalous results for rigid and hot mix asphalt overlay designs using the above overlay design procedures. These cases sometimes occur with strong subgrade soil or with existing composite pavements, i.e., flexible over rigid pavement. Engineering judgment should be applied to ensure adequate performance of the overlay, regardless of type, for the particular pavement and design conditions.

CHAPTER 5. PAVEMENTS FOR LIGHT AIRCRAFT

500. GENERAL. Pavements for light aircraft are defined as those intended to serve aircraft with gross weights of less than 30,000 lbs (13 000 kg). Aircraft of this size are usually engaged in nonscheduled activities such as agricultural, instructional, or recreational flying. Pavements designed to serve these aircraft may be flexible or rigid-type pavements. The design of pavements serving aircraft of 30,000 pounds (13 000 kg) gross weight or more should be based on the criteria contained in Chapter 3 of this publication. Some areas of airports serving light aircraft may not require paving. In these areas the development of an aggregate-turf or turf surface may be adequate for limited operations of these light aircraft. Aggregate-turf surfaces are constructed by improving the stability of a soil with the addition of aggregate prior to development of the turf. Aggregate-turf construction is covered in some detail in the latter part of this chapter. Information on stabilization of soils can be found in Chapter 2 of this circular and in AC 150/5370-10.

501. TYPICAL SECTIONS. Typical cross sections for light aircraft pavements are shown in Figure 5-1. No distinction is made between critical and noncritical pavement sections for pavements serving light aircraft.

502. FLEXIBLE PAVEMENT MATERIALS. Flexible pavements for light aircraft are composed of hot mix asphalt surfacing, base course, subbase and prepared subgrade. The functions of these layers and applicable specifications are discussed below:

a. Hot Mix Asphalt Surfacing. The function of the hot mix asphalt surface or wearing course is the same as discussed earlier in Chapter 3. Specifications covering the composition and quality of hot mix asphalt mixtures is given in Item P-401, Plant Mix Bituminous Mixtures. Note that under certain conditions state highway hot mix asphalt mixtures may be used for pavements intended to serve aircraft weighing 12,500 pounds (5 700 kg) or less.

b. Base Course. As in heavy load pavements, the base course is the primary load carrying component of a flexible pavement. Specifications covering materials suitable for use as base courses for light load pavements are as follows:

(1) Item P-208 - Aggregate Base Course
(2) Item P-209 - Crushed Aggregate Base Course
(3) Item P-210 - Caliche Base Course
(4) Item P-211 - Lime Rock Base Course
(5) Item P-212 - Shell Base Course
(6) Item P-213 - Sand-Clay Base Course
(7) Item P-301 - Soil Cement Base Course
(8) Item P-304 - Cement Treated Base Course
(9) Item P-306 - Econocrete Subbase Course
(10) Item P-401 - Plant Mix Bituminous Pavements

Note: Use of some of the above materials in areas where frost penetrates into the base course may result in some degree of frost heave and/or may require restricted loading during spring thaw.

c. Subbase Course. A subbase course is usually required in flexible pavement except those on subgrades with a CBR value of 20 or greater (usually GW or GP type soils). Materials conforming to specification Item P-154 may be used as subbase course. Also any items listed above in paragraph 83b may be used as subbase course if economy and practicality dictate. Since the loads imposed on these pavements are much less than those on pavements designed for heavier aircraft, compaction control for base and subbase layers should be based on ASTM D 698, Tests for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures, Using 5.5-pound (2.5 kg) Rammer and 12-inch (300 mm) Drop.

d. Stabilized Base and Subbase. Stabilized base and subbase courses may be used in light load pavements. Reduced thicknesses of base and subbase may result. Thickness equivalencies for stabilized materials are given in Chapter 3.

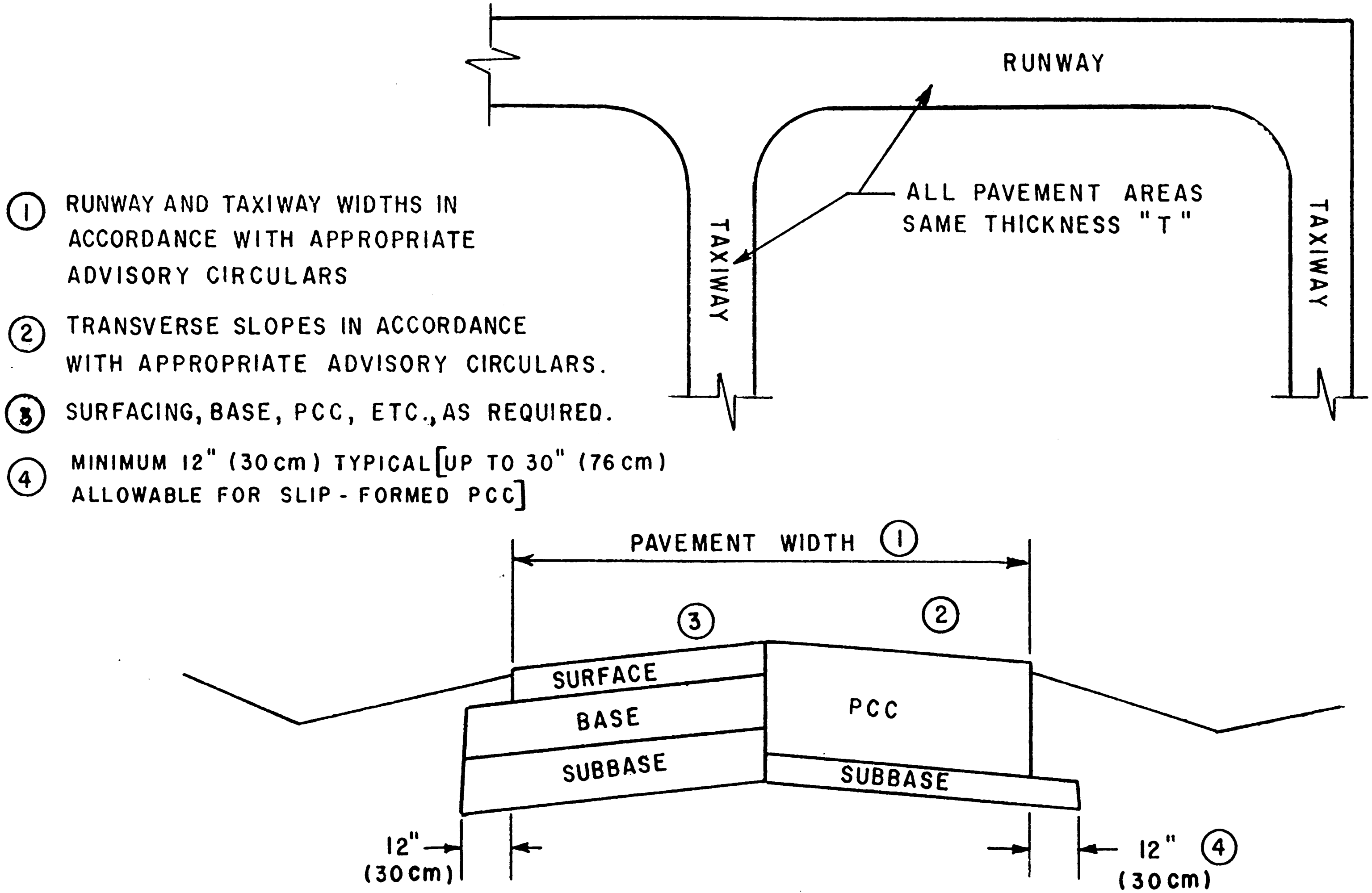


FIGURE 5-1. TYPICAL SECTIONS FOR LIGHT AIRCRAFT PAVEMENTS

e. Subgrade. Subgrade materials should be compacted in accordance with Item P-152 to the depths shown on Table 5-1.

TABLE 5-1. SUBGRADE COMPACTION REQUIREMENTS FOR LIGHT LOAD FLEXIBLE PAVEMENTS

Design Aircraft Gross Weight lbs.	Noncohesive Soils Depth of Compaction (in.)				Cohesive Soils Depth of Compaction (in.)			
	100%	95%	90%	85%	95%	90%	85%	80%
12,500 or less	6	6-9	9-18	18-24	4	4-8	8-12	12-15
12,501 or more	8	8-12	12-24	24-36	6	6-9	9-12	12-15

Notes:
1. Noncohesive soils, for the purpose of determining compaction control, are those with a plasticity index (P.I.) of less than 6.
2. Tabulated values denote depths below the finished subgrade above which densities should equal or exceed the indicated percentage of the maximum dry density as specified in Item P-152.
3. The subgrade in cut areas should have natural densities shown or should (a) be compacted from the surface to achieve the required densities, (b) be removed and replaced at the densities shown, or (c) when economics and grades permit, be covered with sufficient select or subbase material so that the uncompacted subgrade is at a depth where the in-place densities are satisfactory.
4. For intermediate aircraft weights use linear interpolation.
5. For swelling soils refer to paragraph 314.
6. 1 inch = 25.4 mm, 1 lb. = 0.454 kg

503. FLEXIBLE PAVEMENT DESIGN. Figure 5-2 shows the pavement thickness requirements for aircraft weighing up to 30,000 pounds (13 000 kg) gross weight. The pavement thickness determined from Figure 5-2 should be used on all areas of the airport pavement. No reduction in thickness should be made for "noncritical" areas of pavements. For very light load pavements, the design should also consider the weight of maintenance and fueling equipment. It is possible that these types of equipment may require a thicker pavement section than the aircraft.

a. Total Pavement Thickness. Use of the curve requires a CBR value for the subgrade and the gross weight of the design aircraft. The preferred method of establishing the subgrade CBR is by testing. The testing procedures described in Chapter 3 should also be applied to light load pavements. In instances where CBR tests are not practical, the values listed in Table 2-3 may be used.

b. Thickness of Surfacing and Base. The thickness of surfacing and base is determined by using the CBR-20 line. The difference between the total pavement thickness required and the CBR-20 line thickness, composed of surfacing and base, yields the thickness of subbase. Note that the minimum thickness of hot mix asphalt surfacing over a granular base is 2 inches (50 mm).

c. Thin Lifts. The reason for the minimum surfacing thickness is that layers thinner than 2 inches (50 mm) are difficult to place and compact on granular bases. Hot mix asphalt surfacing thickness of less than 2 inches (50 mm) is permissible on stabilized base materials if proper laydown and compaction can be achieved. The base course thicknesses in Figure 5-2 range from 3 inches (75 mm) to 6 inches (150 mm) while the subbase thicknesses vary from 0-14 inches (0-355 mm). In some instances difficulties may be encountered in compacting thin bases or subbases. In these cases the base or subbase thicknesses may be increased to facilitate construction even though the additional thickness is not needed for structural capacity.

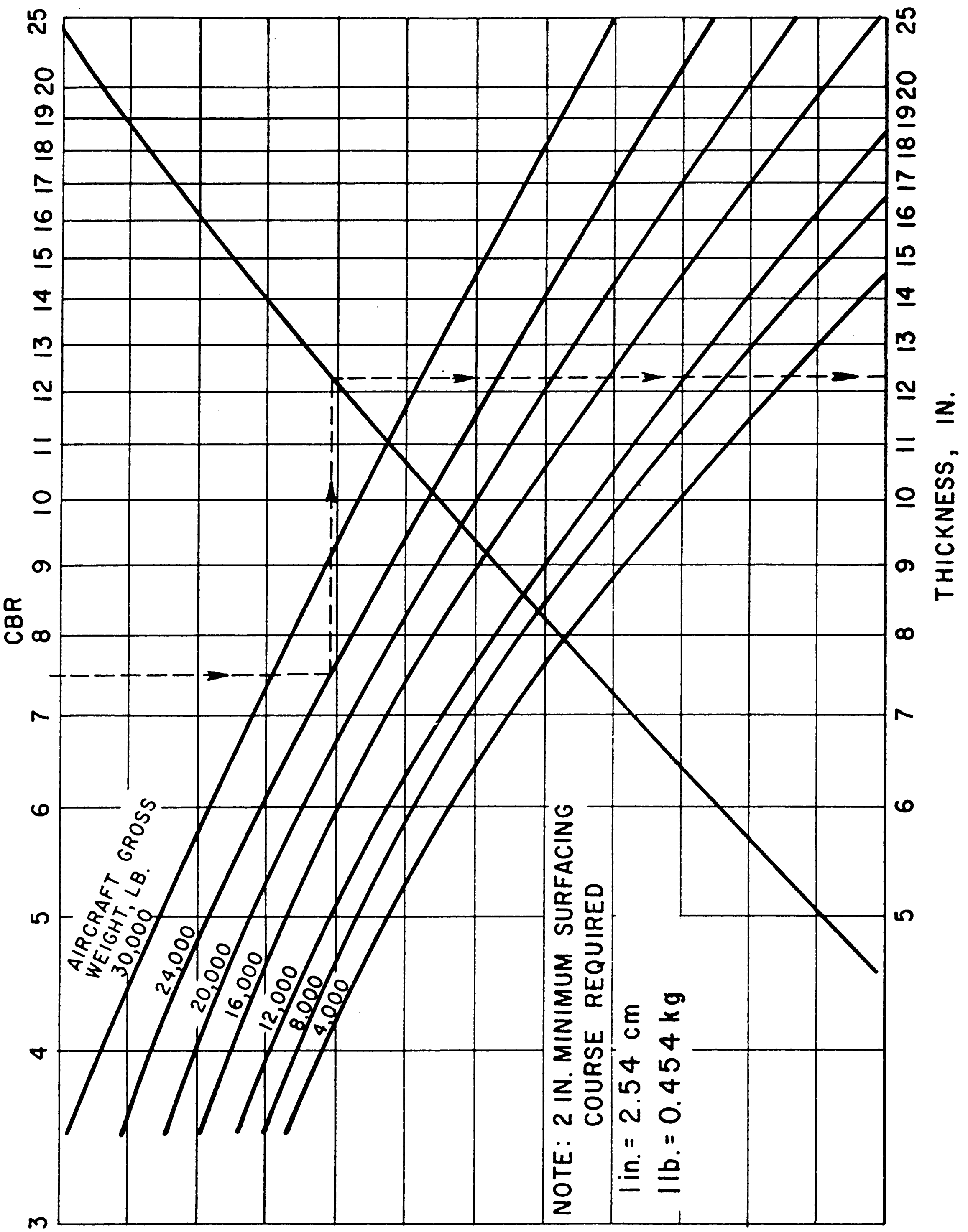


FIGURE 5-2. DESIGN CURVES FOR FLEXIBLE PAVEMENTS - LIGHT AIRCRAFT

d. Example. As an example of the use of Figure 5-2, assume a pavement is to be designed for the following conditions.

Aircraft gross weight = 20,000 lbs. (9100 kg)
Subgrade CBR = 7

(1) Total Pavement Thickness. Enter the upper abscissa of Figure 5-2 with the subgrade CBR value of 7. Make a vertical projection downward to the aircraft gross weight line of 20,000 lbs. (9100 kg). At the point of intersection of the vertical projection and the aircraft gross weight line, make a horizontal projection to the pivot line. At the point of intersection of the horizontal projection and the pivot line, make a vertical projection down to the lower abscissa and read the total pavement thickness required, in this example 11.8 inches (300 mm).

(2) Thickness of Surfacing and Base. To determine the thickness of surfacing and base proceed as in the steps above using a CBR value of 20. In this example, a thickness of 5 inches (127 mm) is read on the lower abscissa. This represents the combined thickness of surfacing and base.

(3) Final Design Section. The design section would thus consist of 2 inches (50 mm) of hot mix asphalt surfacing, 3 inches (75 mm) of base, and 7 inches (178 mm) of subbase. Should difficulties be anticipated in compacting the 3-inch (75 mm) base course, the base course thickness should be increased. The thickness increase can be accomplished by substituting some of the subbase material with base course. If base material is substituted for subbase material, a thickness credit can be taken. The thickness credit should be determined using the equivalency factors given in Table 3-7.

e. Omission of Hot Mix Asphalt Surfacing. Under certain conditions, it may be desirable to utilize a bituminous surface treatment on a prepared base course in lieu of hot mix asphalt. In such instances the strength of the pavement is furnished by the base, subbase, and subgrade. Additional base course thickness will be necessary to make up for the missing surface course. Additional base should be provided at a ratio of 1.2 to 1.6 inches (30 - 41 mm) of base for each 1 inch (25 mm) of surfacing.

f. Full-Depth Asphalt Pavements. Pavements to serve light aircraft may be constructed of full-depth asphalt using the criteria specified in paragraph 323. The Asphalt Institute has published guidance on the design of full depth asphalt pavements for light aircraft in Information Series No. 154. See Appendix 4. Use of the Asphalt Institute method of design for full-depth asphalt pavements requires approval on a case-by-case basis.

g. Local Materials. Since the base and subbase course materials discussed in Chapter 3 are more than adequate for light aircraft, full consideration should be given to the use of locally available, less expensive materials. These locally available materials may be entirely satisfactory for light load pavements. These materials may include locally available granular materials, soil aggregate mixtures, or soils stabilized with portland cement, bituminous materials, or lime. The designer is cautioned, however, if the ultimate design of the pavement is greater than 30,000 lbs (13 000 kg), higher quality materials should be specified at the outset.

504. RIGID PAVEMENT MATERIALS. Rigid pavements for light aircraft are composed of portland cement concrete surfacing, subbase, and prepared subgrade. The functions of these layers and applicable specifications are discussed below:

a. Portland Cement Concrete. Specifications concerning the quality and placement of portland cement concrete should be in accordance with Item P-501, Portland Cement Concrete Pavement. Local state highway specifications for paving quality concrete may be substituted for Item P-501 if desired.

b. Subbase. Rigid pavements designed to serve aircraft weighing between 12,500 pounds (5700 kg) and 30,000 pounds (13000 kg) will require a minimum subbase thickness of 4 inches (100 mm) except as shown in Table 3-4 of Chapter 3. No subbase is required for designs intended to serve aircraft weighing 12,500 pounds (5 700 kg) or less, except when soil types OL, MH, CH or OH are encountered. When the above soil types are present, a minimum 4-inch (100 mm) subbase should be provided. The materials suitable for subbase courses are covered in Item P-154, Subbase Course.

c. Subgrade. Subgrade materials should be compacted in accordance with Item P-152 to the following depths. For cohesive soils used in fill sections, the entire fill shall be compacted to 90 percent maximum density. For cohesive soils in cut sections, the top 6 inches (150 mm) of the subgrade shall be compacted to 90% maximum density. For noncohesive soils used in fill sections, the top 6 inches (150 mm) of fill shall be compacted to 100 percent maximum density, and the remainder of the fill shall be compacted to 95 percent maximum density. For cut sections in noncohesive soils, the top 6 inches (150 mm) of subgrade shall be compacted to 100 percent maximum density and the next 18 inches (460 mm) of subgrade shall be compacted to 95 percent maximum density. For treatment of swelling soils refer to paragraph 314.

505. RIGID PAVEMENT THICKNESS. No design curves for light duty rigid pavements are presented since there are only two thickness requirements. Rigid pavements designed to serve aircraft weighing 12,500 pounds (5 700 kg) or less should be 5 inches (127 mm) thick. Those designed to serve aircraft weighing between 12,501 pounds (5 700 kg) and 30,000 pounds (13 000 kg) should be 6 inches (150 mm) thick.

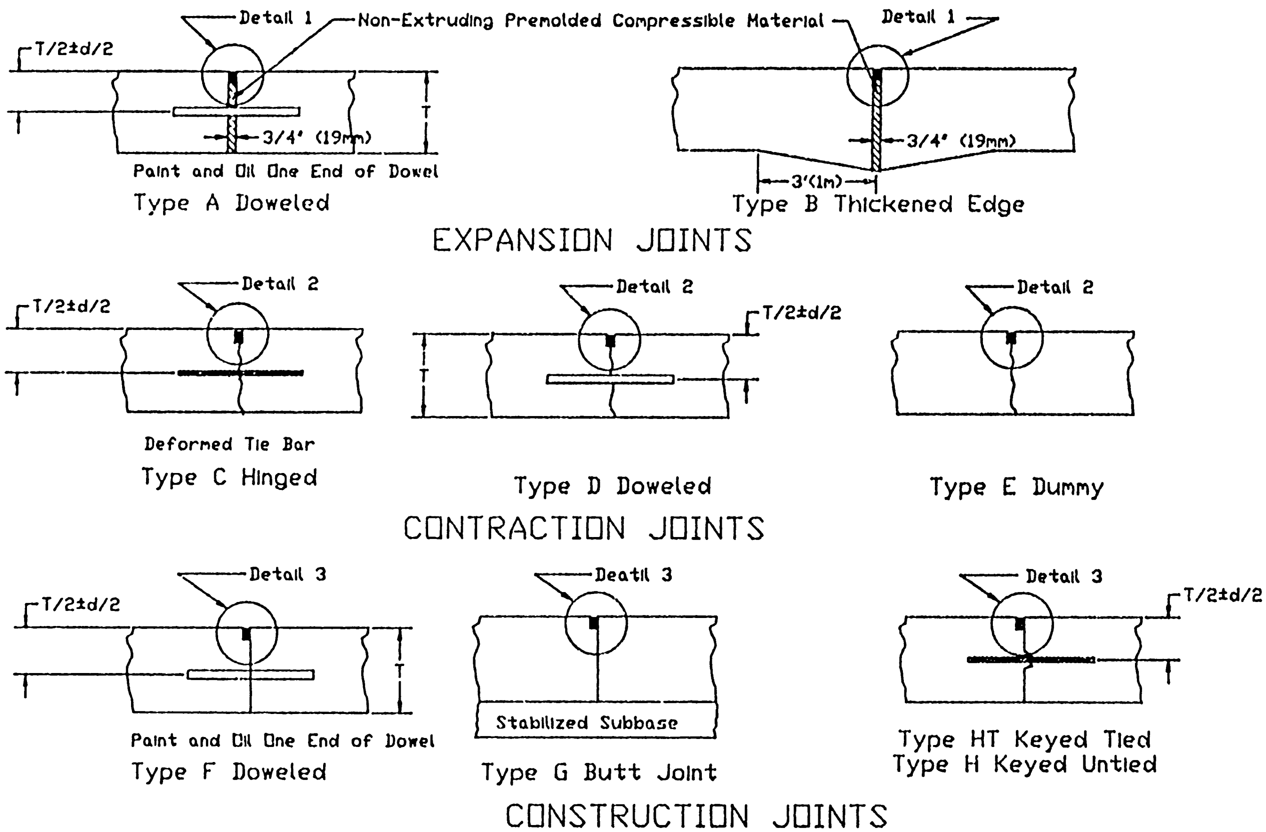
a. Jointing of Light Load Rigid Pavements. The maximum spacing of joints for light load rigid pavements should be 12.5 feet (3.8 m) for longitudinal joints and 15 feet (4.6 m) for transverse joints. Jointing details are shown in Figure 5-3. Note that several differences exist between light load and heavy load rigid pavement joints. Such as butt-type construction and expansion joints are permitted when an asphalt or cement stabilized subbase is provided. Also half round keyed joints are permitted even though the slab thicknesses are less than 9 inches (230 mm). Odd-shaped slabs should be reinforced with 0.05% steel in both directions. Odd-shaped slabs are defined as slabs which are not rectangular in shape or rectangular slabs which length-to-width ratios exceed 1.25. Two recommended joint layout patterns are shown in Figure 5-4 for 60 foot (18 m) and Figure 5-5 for 50 foot (15 m) wide pavements. The concept behind the jointing patterns shown is the creation of a "tension ring" around the perimeter of the pavement to hold joints within the interior of the paved area tightly closed. A tightly closed joint will function better than an open joint. The last three contraction joints and longitudinal joints nearest the free edge of the pavement are tied with #4 deformed bars, 20 inches (510 mm) long, spaced at 36 inches (1 m) center to center. At the ends of the pavement and in locations where aircraft or vehicular traffic would move onto or off the pavement, a thickened edge should be constructed. The thickened edge should be 1.25 times the thickness of the slab and should taper to the slab thickness over a distance of 3 feet (1 m).

506. AGGREGATE TURF. Aggregate-turf differs from normal turf in that the stability of the underlying soil is increased by the addition of granular materials prior to establishment of the turf. The objective of this type of construction is to provide a landing areas that will not soften appreciably during wet weather and yet has sufficient soil to promote the growth of grass. Aggregate-turf should be considered only for areas designed to serve aircraft having gross weights of 12,500 pounds (5 700 kg) or less.

a. Materials. Construction details and material requirements are covered in Item P-217, Aggregate--Turf Pavement. A minimum CBR of 20 is recommended for aggregate/soil layers.

b. Thickness. The thickness to be stabilized with the granular materials varies with the type of soil and the drainage and climatic conditions. The total thickness of aggregate stabilized soil should be read directly from the thickness scale of Figure 5-2 using the CBR of the subgrade, disregard note concerning surfacing course.

507. OVERLAYS. Overlays of pavements intended to serve light aircraft are designed in the same manner as overlays for heavy aircraft.



Notes:
1. For Details 1, 2, &3 See Next Page.
2. All Dowels 3/4" (19mm) Dia, 18" (460 mm) Long, Spaced 12" (300 mm) on Centers
3. All Tie Bars No. 4 Deformed Bars 20" (510 mm) Long, Spaced 36" (0.9m) on Centers

FIGURE 5-3. JOINTING DETAIL FOR LIGHT LOAD RIGID PAVEMENTS

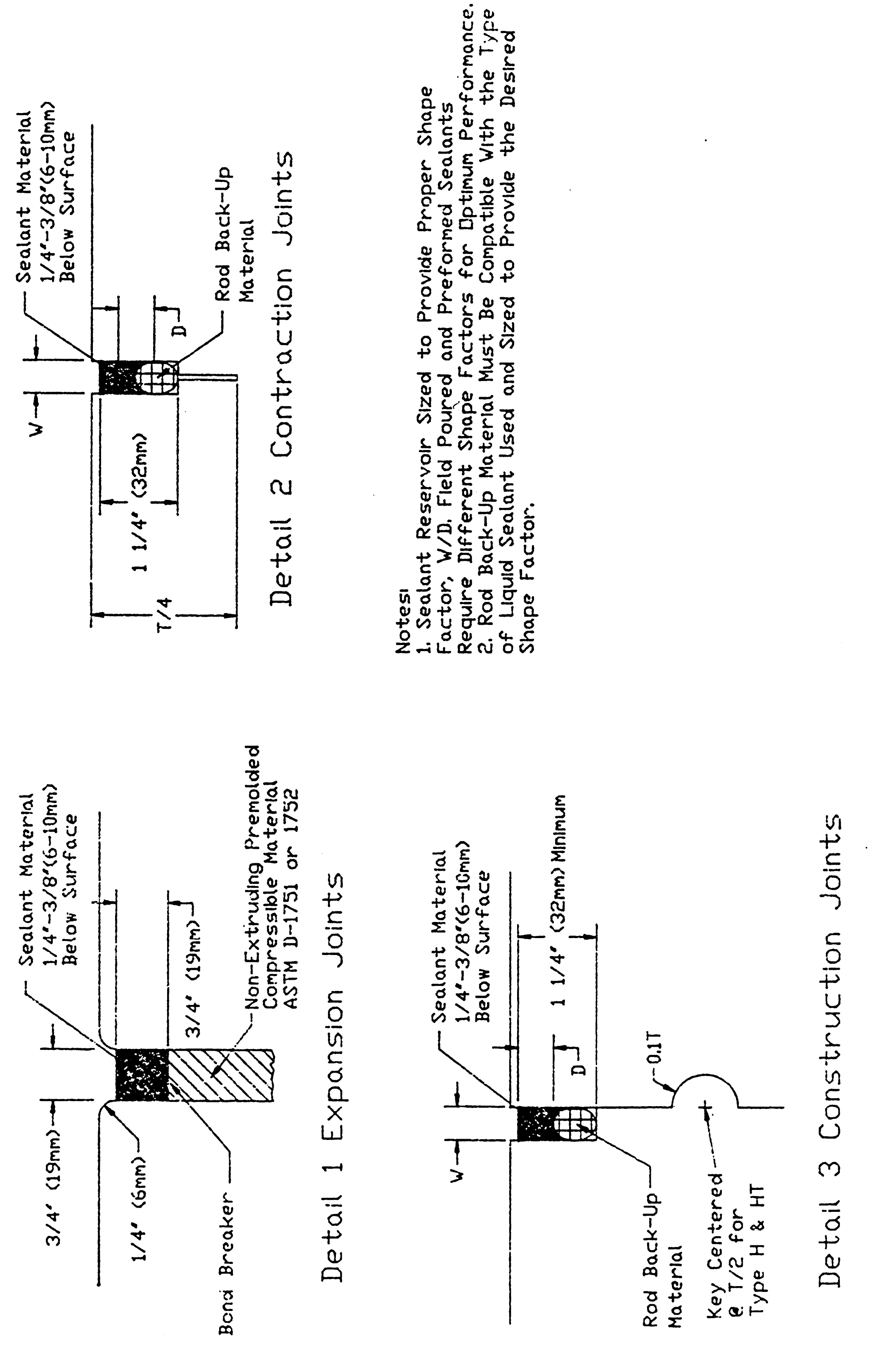


FIGURE 5-4. CON'T JOINTING DETAILS FOR LIGHT LOAD RIGID PAVEMENTS

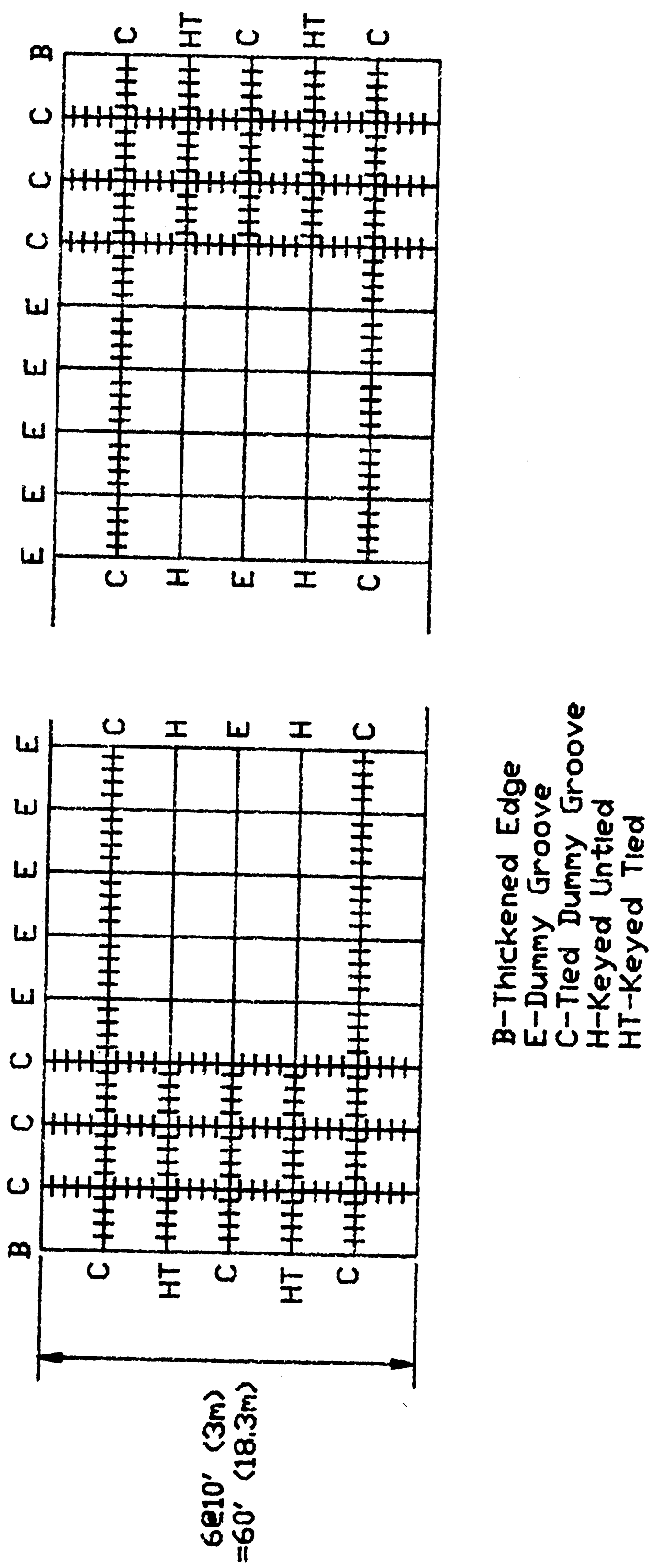


FIGURE 5-5. JOINT LAYOUT PATTERNS FOR LIGHT LOAD RIGID PAVEMENTS - 60 FEET WIDE

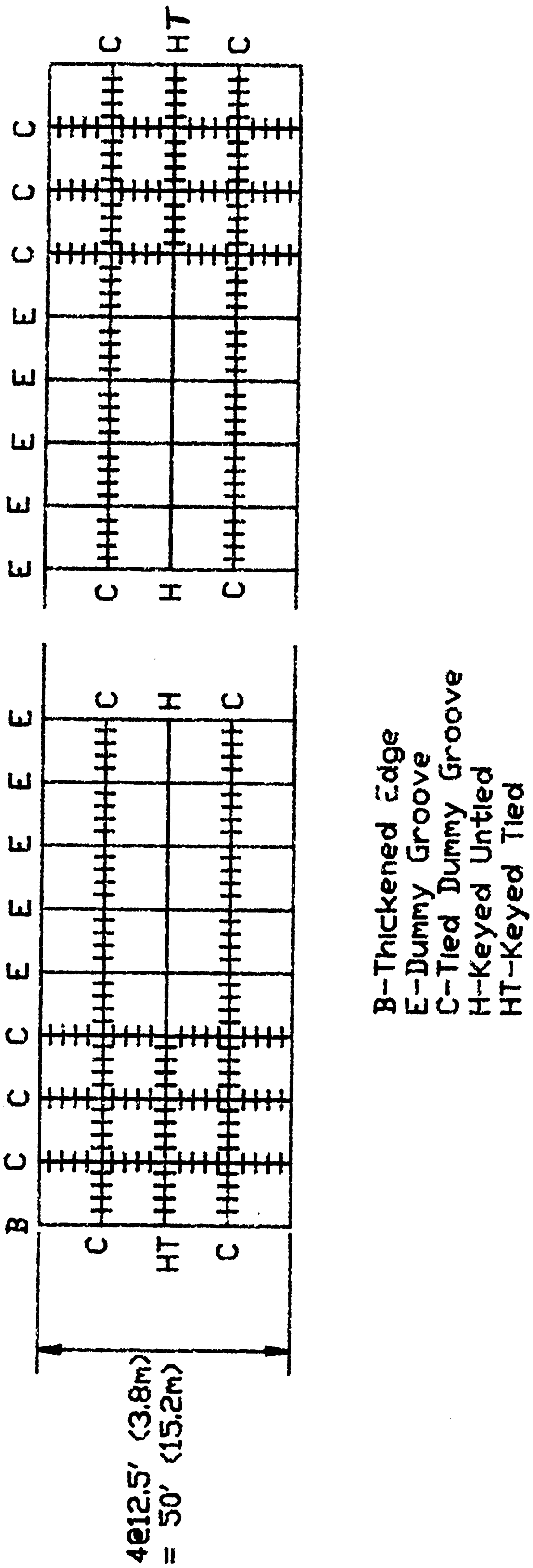


FIGURE 5-6. JOINT LAYOUT PATTERNS FOR LIGHT LOAD RIGID PAVEMENTS - 50 FEET WIDE

CHAPTER 6. PAVEMENT EVALUATION

600. PURPOSES OF PAVEMENT EVALUATION. Airport pavement evaluations are necessary to assess the ability of an existing pavement to support different types, weights, or volumes of aircraft traffic. The load carrying capacity of existing bridges, culverts, storm drains, and other structures should also be considered in these evaluations. Evaluations may also necessary to determine the condition of existing pavements for use in the planning or design of improvements to the airport. Evaluation procedures are essentially the reversal of design procedures. This chapter covers the evaluation of pavements for all weights of aircraft.

601. EVALUATION PROCESS. The evaluation of airport pavements should be a methodical step-by-step process. The recommended steps in the evaluation process are given in the subsequent paragraphs regardless of the type of pavement.

a. Records Research. A thorough review of construction data and history, design considerations, specifications, testing methods and results, as-built drawings, and maintenance history should be performed. Weather records and the most complete traffic history available are also parts of a usable records file.

b. Site Inspection. The site in question should be visited and the condition of the pavements noted by visual inspection. This should include, in addition to the inspection of the pavements, an examination of the existing drainage conditions and drainage structures at the site. Evidence of the adverse effects of frost action, swelling soils, reactive aggregates, etc. should also be noted. The principles set forth in Chapter 2 of this circular and in AC 150/5320-5, Airport Drainage, apply.

c. Sampling And Testing. The need for and scope of physical tests and materials analyses will be based on the findings made from the site inspection, records research, and type of evaluation. A complete evaluation for detailed design will require more sampling and testing than, for example, an evaluation intended for use in a master plan. Sampling and testing is intended to provide information on the thickness, quality, and general condition of the pavement elements.

(1) Direct Sampling Procedures. The basic evaluation procedure for planning and design will be visual inspection and reference to the FAA design criteria, supplemented by the additional sampling, testing, and research which the evaluation processes may warrant. For relatively new pavement constructed to FAA standards and without visible sign of wear or stress, strength may be based on inspection of the FAA Form 5100-1, Airport Pavement Design, and the as-constructed sections, with modification for any material variations or deficiencies of record. Where age or visible distress indicates the original strength no longer exists, further modification should be applied on the basis of judgment or a combination of judgment and supplemental physical testing. For pavements which consist of sections not readily comparable to FAA design standards, evaluation should be based on FAA standards after material comparison and equivalencies have been applied.

(2) Nondestructive Testing. Several methods of nondestructive testing (NDT) of pavements are available. For purposes of this discussion, NDT means of observing pavement response to a controlled loading. provides a means of evaluating pavements which tends to remove some of the subjective judgment needed in other evaluation procedures. FAA Advisory Circular 150/5370-11, Use of Nondestructive Testing Devices in the Evaluation of Airport Pavements, contains guidance on nondestructive testing. The major advantages of nondestructive testing are: the pavement is tested in place under actual conditions of moisture, density, etc.; the disruption of traffic is minimal; and the need for destructive tests is minimized. Research efforts are continuing in the area of nondestructive testing to broaden its application. Several different NDT procedures are available in addition to that described in AC 150/5370-11. These other procedures may be used when approved by FAA.

d. Other Evaluation Tools. There are a number of other tools available to assist the evaluator. These tools include: pavement condition index, ground penetrating radar, infrared thermography, etc.

(1) Pavement Condition Index. The determination of the Pavement Condition Index (PCI) is often a useful tool in the evaluation of airport pavements. The PCI is a numerical rating of the surface condition of a pavement and is a measure of functional performance with implications of structural performance. PCI values range from 100 for a pavement with no defects to 0 for a pavement with no remaining functional life. The index is useful in describing distress and comparing pavements on an equal basis. Advisory Circular 150/5380-6, Guidelines and Procedures for Maintenance of Airport Pavements, contains detailed information on PCI surveys.

(2) **Ground Penetrating Radar.** Ground penetrating radar can be useful in studying subsurface conditions nondestructively. Ground penetrating radar depends on differences in dielectric constants to discriminate between materials. The technique is sometimes used to locate voids or foreign objects, such as, abandoned fuel tanks, tree stumps, etc. in embankments.

(3) **Infrared Thermography.** Infrared thermography is a nondestructive testing procedure whereby differences in infrared emissions are observed allowing certain physical properties of the pavement to be determined. Infrared thermography is purportedly capable of detecting delaminations in bonded rigid overlay pavements and in reinforced rigid pavements.

e. Evaluation Report. The analyses, findings, and test results should be incorporated in an evaluation report which becomes a permanent record for future reference. While evaluation reports need not be in any particular form, it is recommended that a drawing identifying limits of the evaluation be included. Analysis of information gained in the above steps should culminate in the assignment of load carrying capacity to the pavement sections under consideration. When soil, moisture, and weather conditions conductive to detrimental frost action exist, an adjustment to the evaluation may be required.

602. FLEXIBLE PAVEMENTS. Evaluation of flexible pavements requires, as a minimum, the determination of the thickness of the component layers, and the CBR of the subgrade.

a. Layer Thicknesses. The thickness of the various layers in the flexible pavement structure must be known in order to evaluate the pavement. Thicknesses may be determined from borings or test pits. As-built drawings and records can also be used to determine thicknesses if the records are sufficiently complete and accurate.

b. Subgrade CBR. Laboratory CBR tests should be performed on soaked specimens in accordance with ASTM D 1883, Bearing Ratio of Laboratory-Compacted Soils. Field CBRs should be performed in accordance with the procedure given in The Asphalt Institute Manual Series 10 (MS-10), Soils Manual. Field CBR tests on existing pavements less than 3 years old may not be representative unless the subgrade moisture content has stabilized. The evaluation process assumes a soaked CBR is and will not give reliable results if the subgrade moisture content has not reached the ultimate in situ condition. In situations where it is impractical to perform laboratory or field CBR tests, an estimate of CBR based on soil classification is possible. Table 2-3 may be used to estimate CBR on the basis of the Unified Soil Classification System. Prior to adoption of the Unified Soil Classification System, soils were classified using the FAA classification system. Some old records may contain data using the FAA classification system. Figure 6-1 shows the approximate relationship between the FAA soil classification and CBR.

c. Material Comparisons and Equivalencies. For the purposes of design and evaluation, flexible pavements are assumed to be constructed of asphaltic concrete surfacing, granular base, and granular subbase courses of a predetermined quality. When the materials in a pavement structure to be evaluated are at variance with these assumptions, the materials have to be compared and equated to a standard section. The nonstandard sections after conversion have to be checked for load carrying capacity based on the following considerations:

(1) Total pavement section thickness.
(2) Surface plus base course thickness.
(3) Minimum base course thickness.
(4) Minimum surface thickness.

The requirement yielding the lesser strength will control the evaluation.

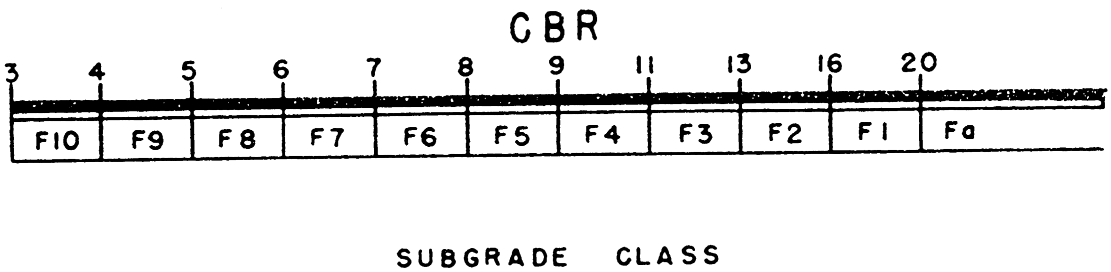


FIGURE 6-1. CBR - FAA SUBGRADE CLASS COMPARISONS

(1) **Subbase And Base Equivalencies.** Equivalency factor ranges shown in Tables 3-6 through and 3-9 for subbase and base are recommended for evaluation purposes. The actual value selected will depend on the composition, quality, and condition of the layer. In instances where experience or physical test results show that other values are valid, they may be used in lieu of the values recommended here. Subbase or base courses should not be assigned a higher equivalency factor than a layer above it in the pavement structure. Conversion of material to a higher classification, such as subbase to base, will not be permitted, except where excess stabilized base course (P-401 or P-304) exists immediately under a flexible surface, in this instance, the stabilized material may be counted as an equal thickness of surface.

(2) **Surfacing.** Broken hot mix asphalt surface course (shrinkage cracks due to age and weathering, without evidence of base failure) shall be evaluated as an equal thickness of nonstabilized base. A hot mix asphalt surface, with limited cracking and well maintained, may justify use of an equivalency between the limits noted.

603. APPLICATION OF FLEXIBLE PAVEMENT EVALUATION PROCEDURES. After all of the evaluation parameters of the existing flexible pavement have been established using the guidance given in the above paragraphs, the evaluation process is essentially the reverse of the design procedure. The design curves presented in Chapter 3 or 5 are used to determine the load carrying capacity of the existing pavement. Required inputs are subgrade and subbase CBR values, thicknesses of surfacing, base and subbase courses and an annual departure level. Several checks must be performed to determine the load carrying capacity of a flexible pavement. The calculation which yields the lowest allowable load will control the evaluation.

a. Total Pavement Thickness. Enter the lower abscissa of the appropriate design curve in Chapter 3 or 5 with the total pavement thickness of the existing pavement. Make a vertical projection to the annual departure level line. For light load pavements, Chapter 5, a single pivot line is used. At the point of intersection between the vertical projection and the departure level line, or single pivot line in the case of light load pavements, make a horizontal projection across the design curve. Enter the upper abscissa with the CBR value of the subgrade. Make a vertical projection downward until it intersects the horizontal projection made previously. The point of intersection of these two projections will be in the vicinity of the load lines on the design curves. An allowable load is read by noting where the intersection point falls in relation to the load lines.

b. Thickness of Surfacing and Base. The combined thickness of surfacing and base must also be checked to establish the load carrying capacity of an existing flexible pavement. This calculation requires the CBR of the subbase, the combined thickness of surfacing and base and the annual departure level as inputs. The procedure is the same as that described in subparagraph a above, except that the subbase CBR and combined thickness of surfacing and base are used to enter the design curves.

c. Minimum Base Course Thickness. The thickness of the existing base course should be compared with the minimum base course thicknesses Table 3-4 or from Figure 5-2. Notice that the minimum base course thickness is 4 inches (100 mm) for heavy load pavements and 3 inches (75 mm) for light load pavements. If there is a deficiency in the thickness of the existing base course, the pavement should be closely monitored for signs of distress. The formulation of plans for overlaying the pavement to correct the deficiency should be considered.

d. Minimum Surface Thickness. The thickness of the existing surface course should be compared with that shown on the appropriate design curve. If the existing surface course is thinner than that given on the design curve, the pavement should be closely observed for surface failures. It is recommended that correction of the deficiency in surfacing thickness be considered.

604. RIGID PAVEMENTS. Evaluation of rigid pavements requires, as a minimum, the determination of the thickness of the component layers, the flexural strength of the concrete, and the modulus of subgrade reaction.

a. Layer Thicknesses. The thickness of the component layers is sometimes available from construction records. Where information is not available or of questionable accuracy, thicknesses may be determined by borings or test pits in the pavement.

b. Concrete Flexural Strength. The flexural strength of the concrete is most accurately determined from test beams sawed from the existing pavement and tested in accordance with ASTM C 78. Quite often this method is impractical as sawed beams are expensive to obtain and costs incurred in obtaining sufficient numbers of beams to establish a representative sample is prohibitive. Construction records, if available, may be used as a source of concrete flexural strength data. The construction data will probably have to be adjusted for age as concrete strength increases with time. Strength-age relationships can be found in Portland Cement Association, Engineering Bulletin, Design of Concrete Airport Pavement.

(1) Correlations With Other Strength Tests. Correlations between concrete flexural strength and other concrete strength tests are available. It should be noted that correlations between flexural strength and other strength tests are approximate and considerable variations are likely.

(i) Tensile Split Strength. An approximate relationship between concrete flexural strength and tensile splitting strength (ASTM C 496) exists and can be computed by the following formula:

$$R = 1.02(f'_t + 200 \text{ psi})$$

where:

R = flexural strength, psi

f'_t = tensile split strength, psi

Note: For conversions in metric units the above formula remains the same, except the +200 psi constant should be changed to +1.38 Mpa.

(ii) Compressive Strength. Flexural strength can be estimated from compressive strength (ASTM C 39) using the formula below:

$$R = 9\sqrt{f'_c}$$

where:

R = flexural strength

f'_c = compressive strength

c. **Modulus of Subgrade Reaction.** The modulus of subgrade reaction is determined by plate bearing tests performed on the subgrade. These tests should be made in accordance with the procedures established in AASHTO T 222. An important part of the test procedure for determining the subgrade reaction modulus is the correction for soil saturation which is contained in the prescribed standard. The normal application utilizes a correction factor determined by the consolidation testing of samples at in situ and saturated moisture content. For evaluation of older pavement, where evidence exists that the subgrade moisture has stabilized or varies through a limited range, the correction for saturation is not necessary. If a field plate bearing test is not practical, the modulus of subgrade reaction may be determined by nondestructive testing, or estimated by using Table 2-3 in Chapter 2 of this circular. Fortunately, pavement evaluation is not too sensitive to the modulus of subgrade reaction.

(1) **Adjustment For Subbase.** An adjustment to the modulus of subgrade reaction will be required if a subbase exists beneath the existing pavement. The thickness of the subbase is required to calculate an adjusted k value. The subbase thickness can be determined from construction records or from borings. The guidance contained in Chapter 3, Section 3 should be used in assigning a k value to a subbase.

605. APPLICATION OF RIGID PAVEMENT EVALUATION PROCEDURES. The evaluation of rigid pavements for aircraft weighing in excess of 30,000 pounds (13 600 kg) requires concrete flexural strength, k value of the foundation, slab thickness, and annual departure level as inputs. The rigid pavement design curves in Chapter 3 are used to establish load carrying capacity. The design curves are entered on the left ordinate with the flexural strength of the concrete. A horizontal projection is made to the k value of the foundation. At the point of intersection of the horizontal projection and the k line, a vertical projection is made into the vicinity of the load lines. The slab thickness is entered on the appropriate departure level scale on the right side of the chart. A horizontal projection is made from the thickness scale until it intersects the previous vertical projection. The point of intersection of these projections will be in the vicinity of the load lines. The load carrying capacity is read by noting where the intersection point falls in relation to the load lines.

606. USE OF RESULTS. If the evaluation is being used for planning purposes and the existing pavement is found to be deficient in accordance with the design standards given in Chapter 3 or 5, the sponsor should be notified as to the deficiency and consideration should be given to corrective action. If the evaluation is being used a part of the design for a project to reconstruct or upgrade the facility, the procedures given in Chapters 3, 4, or 5 should be used to design the reconstruction or overlay project. In this instance the main concern is not the load carrying capacity but rather the difference between the existing pavement structure and the section that is needed to support forecast traffic.

607. REPORTING PAVEMENT STRENGTH. The International Civil Aviation Organization (ICAO) developed a standardized method of reporting airport pavement strength known as the Aircraft Classification Number/Pavement Classification Number ACN/PCN. This method of reporting is based on the concept of reporting strength in terms of a standardized equivalent single wheel load. This method of reporting pavement strength is discussed in FAA Advisory Circular 150/5335-5, Standardized Method of Reporting Airport Pavement Strength - PCN.

APPENDIX 1. ECONOMIC ANALYSIS

1. **BACKGROUND.** The information presented in this appendix was developed from research report DOT/FAA/RD-81/78. The cost data used are probably not current, however, the principles and procedures are applicable. An example is given for illustrative purposes.

2. **ANALYSIS METHOD.**

a. Present worth or present value economic analyses are considered the best methods for evaluating airport pavement design or rehabilitation alternatives. A discount rate of 4 percent is suggested together with an analysis period of 20 years. Residual salvage values should be calculated on the straight-line depreciated value of the alternative at the end of the analysis period. The initial cost and life expectancy of the various alternatives should be based on the engineer's experience with consideration given to local materials, environmental factors and contractor capability.

b. The basic equation for determining present worth is shown below:

$$PW = C + \sum_{i=1}^{m} M_i \left(\frac{1}{1+r}\right)^{n_i} - S\left(\frac{1}{1+r}\right)^{z}$$

Where:

PW = Present Worth
C = Present Cost of initial design or rehabilitation activity
m = Number of maintenance or rehabilitation activities
M_i = Cost of the ith maintenance or rehabilitation alternative in terms of present costs, i.e., constant dollars
r = Discount rate (four percent suggested)
n_i = Number of years from the present of the ith maintenance or rehabilitation activity
S = Salvage value at the end of the analysis period
z = Length of analysis period in years (20 years suggested)

The term: $\left(\frac{1}{1+r}\right)^n$

is commonly called the single payment present worth factor in most engineering economic textbooks. From a practical standpoint, if the difference in the present worth of costs between two design or rehabilitation alternatives is 10 percent or less, it is normally assumed to be insignificant and the present worth of the two alternatives can be assumed to be the same.

3. **STEP BY STEP PROCEDURE.** The information presented in this appendix is intended to demonstrate how to calculate cost comparisons for airport pavement alternatives using the present worth method. The following is a step by step procedure illustrating the analysis method.

a. Identify and record key project descriptions such as:

(1) Project Number and Location

(2) Type of Facility

(3) Design Aircraft

(4) Annual Departure of Design Aircraft

(5) Subgrade Strength

b. If appropriate, determine the condition of existing pavement and record data such as:

(1) Existing Pavement Layers (thicknesses, etc.)

(2) Condition of Pavement (description of distress, pavement condition index, P.C.I., [see AC 150/5380-6], etc.)

(3) Skid Resistance

(4) Required Thickness of New Pavement

c. Identify what feasible alternatives are available.

d. Determine costs associated with each feasible alternative in terms of present day costs.

(1) Initial Cost

(2) Maintenance

(3) Future Rehabilitation

e. Calculate life-cycle cost for each alternative to be evaluated.

f. Summarize life-cycle costs, length of time required to perform and the chance for success for each alternative.

g. Evaluated the most promising alternatives based on costs, time required, operational constraints, chance for success, etc.

h. If the selection cannot be narrowed to one alternative in the evaluation process, the most promising alternatives should each be bid and the selection made on the basis of the lowest bid.

4. **EXAMPLE PROBLEM - LIGHT-LOAD GENERAL AVIATION AIRPORT.** An example problem is discussed below which illustrates the use of the present worth life-cycle costing techniques described above.

a. A general aviation airport runway is in need of rehabilitation. The existing pavement contains alligator, transverse, and longitudinal cracking. The design aircraft for the facility has a gross weight of 24,000 lbs. (10 890 kg). Using the procedures in Chapter 5 of this circular, a 3 inch (76 mm) thick bituminous overlay is required to rehabilitate the pavement. Pertinent data are presented in the Project Summary.

PROJECT SUMMARY

Location - Muddville, TX	Design Aircraft: 24,000 lbs. (10 890 kg)
Number - A.I.P. 12-34-567	Annual Departures of Design Aircraft: 3,000
Type of Facility: General Aviation Runway length = 3,200 ft. (75 m)	Subgrade Strength: CBR = 4

width = 75 ft. (23 m)

Existing Pavement:

Layer and Type	Thickness	Condition
AC Surface	4in. (102 mm)	Poor
Untreated Base	10in. (254 mm)	Good

Condition of Existing Pavement:

Condition Survey: Alligator cracking, moderate 15% of area
Trans. cracking, moderate, 350'/station
Long. cracking, moderate, 400'/station
P.C.I. = 35

Skid Resistance: Good

Req'd Thickness New Pave. = 18 in. (487 mm) total
2 in. (51 mm) surf.
5 in. (127 mm) base
11 in. (279 mm) subbase

b. Seven rehabilitation alternatives including surface, in-place, and hot-mix recycling are considered feasible. The alternatives under consideration are:

(1) Asphalt-rubber chip seal to delay overlay

(2) Full width 3-inch (76 mm) direct overlay

(3) Surface recycle 1-inch (25 mm) deep + 2-inch (51 mm) overlay

(4) Asphalt-rubber interlayer + 3-inch (76 mm) overlay

(5) Fabric interlayer + 3-inch (76 mm) overlay

(6) Cold recycle with asphalt emulsion 6-inch (152 mm) deep + 2-inch (51 mm) overlay

(7) Hot recycle and rework base

c. The present day costs of various activities associated with these alternatives are estimated as shown in Table 1.

TABLE 1. COSTS OF REHABILITATION ACTIVITIES

Rehabilitation Activity	Cost $/yd^2	$/m^2
Asphalt-Rubber Chip Seal	1.25	(1.50)
Asphalt-Rubber Interlayer	1.25	(1.50)
Fabric Interlayer	1.20	(1.44)
Surface Recycling	0.90	(1.08)
Asphaltic Concrete - 1 in. (25 mm)	1.65	(1.97)
Cold Recycle + 2 in. (51 mm) Overlay	6.60	(7.89)
Hot Recycle + Rework Base	8.10	(9.69)

d. The life-cycle costs for each alternative are calculated. This example shows the calculations for only one alternative, the asphalt-rubber chip seal. The calculations are shown in Table 2. Some of the important aspects of this analysis are discussed further below.

TABLE 2. PRESENT WORTH LIFE-CYCLE COSTING
EXAMPLE 1. ALTERNATIVE 1. ASPHALT-RUBBER CHIP SEAL

Year	Cost, $/yd²	Present Worth Factor, 4%	Present Worth Dollars
0 A-R Chip Seal	1.25	1.0000	1.25
1		0.9615	
2		0.9246	
3 Maintenance	0.25	0.8890	0.22
4 3" Overlay	4.95	0.8548	4.23
5		0.8219	
6		0.7903	
7		0.7599	
8		0.7307	
9		0.7026	
10 Maintenance	0.10	0.6756	0.07
11 Maintenance	0.10	0.6496	0.06
12 Maintenance	0.10	0.6246	0.06
13 Maintenance	0.10	0.6006	0.09
14 Maintenance	0.25	0.5775	0.14
15 1 1/2" Overlay	2.48	0.5553	1.38
16		0.5339	
17		0.5134	
18		0.4936	
19 Maintenance	0.10	0.4746	0.05
20 Maintenance	0.15	0.4564	0.07
Sub Total	9.88		
Salvage Value	-0.71	0.4564	-0.32
Total	9.17		7.300

Note: To convert from $/yd.² to $/m², divide by 0.8361.

(1) The asphalt-rubber chip seal is estimated to delay the need for an overlay for 4 years. In the third year the asphalt-rubber chip seal will need maintenance costing $0.25/yd² ($0.29/m²).

(2) In the fourth year a 3-inch (76 mm) overlay will be required. This overlay will require maintenance starting in the 10th year and will require progressively more maintenance as time goes on. In the 14th year maintenance will reach $0.25/yd² ($0.29/m²).

(3) In the 15th year a 1.5-inch (38mm) leveling course will be required. This leveling course will not require maintenance until the 19th year. Maintenance costs begin to escalate again as time goes on.

(4) The 20th year marks the end of the analysis period. The salvage value of the leveling course is: the ratio of the life remaining/to how long it will last; multiplied by its costs. The leveling course, constructed in the 15th year, is expected to have a life of 7 years. It was used for only 5 years during the analysis period. Thus, the leveling course had 2 years of life remaining at the end of the analysis period. The salvage value is 2/7 x $2.48 = $0.71. Discounting the salvage value to the 20th year yields a salvage value of $0.32. Since the salvage value is an asset rather than a cost, it is shown as a negative cost in Table 2. All other activities are assumed to have no salvage value since their useful lives have been exhausted during the analysis period. In this example, a discount rate of 4% was assumed. The present worth calculations for the other six alternatives should be calculated in a similar fashion.

e. A final summary of all alternatives considered in this example is shown in Table 3. This summary shows initial costs, life-cycle costs, construction times, and the probability for success in percent. This final summary is a convenient method of presenting all alternatives for evaluation. In this example a discount rate of 4% was used in all calculations. Maintenance and need for rehabilitation in future years are the engineer's estimates.

TABLE 3. SUMMARY OF ALTERNATIVES

Alternatives	First Cost $/yd.^{2}	Present Worth Life Cycle $/yd.^{2}	Time	Success Chance for %
#1 Asph-Rub Chip Seal	1.25	7.30	2 days	90
#2 3-in. Direct Overlay	4.95	7.29	5 days	95
#3 Surf. Recycle + Overlay	4.20	6.22	4 days	97
#4 A-R Layer + Overlay	6.20	7.39	4 days	97
#5 Fabric + Overlay	6.15	7.74	4 days	97
#6 Cold Recycle	6.60	7.41	6 days	97
#7 Hot Recycle	8.10	8.46	6 days	99

Note: To convert from $/yd.^{2} to $/m^2, divide by 0.8361.

f. Comparing and ranking the various alternatives shown in Table 3 yields the following results:

TABLE 4. COMPARATIVE RANKING OF ALTERNATIVES

First Cost	Life-Cycle Cost	Time	Chance for Success
#1	#3	#1	#7
#3	#2	#3	#3
#2	#1	#4	#4
#5	#4	#5	#5
#4	#6	#2	#6
#6	#5	#6	#2
#7	#7	#7	#1

The average life-cycle cost of all 7 alternatives is $7.40/yd.^{2} ($8.85/m^2). Adding and subtracting 10% to the average life-cycle cost yields a range of $6.66/yd.^{2} to $8.14/yd.^{2} ($7.97/m^2 to $9.74/m^2). Alternative #3, surface recycling with an overlay, is lowest in life-cycle costs. Life-cycle costs for alternatives #1, 3, 4, 5, and 6 are within the 10% range of the average cost. Alternative #7 is the most costly and exceeds 10% of the average cots. Alternative #3 appears to the be most promising as it ranks high in three of the four categories considered. The decision to select alternative #3 must consider the availability of contractors capable of performing surface recycling and the time required for completion.

5. SUMMARY This appendix presents an economic procedure for evaluating a wide variety of airport pavement design strategies. While the design example addresses a rehabilitation project, the principles are applicable to designs of new pavements as well. Cost data used in the example are out of date and should be updated with more current local costs before individual evaluations leading to strategy selection are undertaken. Whenever possible, local costs should be used in all alternative analyses as local conditions sometimes vary considerably from broad overall averages.

APPENDIX 2. DEVELOPMENT OF PAVEMENT DESIGN CURVES

1. **BACKGROUND.** The pavement design curves presented in this circular were developed using the California Bearing Ratio (CBR) method for flexible pavements and Westergaard edge loading analysis for rigid pavements. The curves are constructed for the gross weight of the aircraft assuming 95% of the gross weight is carried on the main landing gear assembly and the remaining 5% is carried on the nose gear assembly. Aircraft traffic is assumed to be normally distributed across the pavement in the transverse direction. See FAA Research Report No. FAA-RD-74-36, Field Survey and Analysis of Aircraft Distribution of Airport Pavement. Pavements are designed on the basis of static load analysis. Impact loads are not considered to increase the pavement thickness requirements. See FAA Research Report No. FAA-RD-74-39, Pavement Response to Aircraft Dynamic Loads.

a. **Generalized Design Curves.** Generalized design curves are presented in Chapter 3 for single, dual, and dual tandem main landing gear assemblies. These generalized curves apply to families of aircraft rather than particular aircraft. Generally speaking the assumed characteristics are representative of older aircraft. The aircraft characteristics assumed for each landing gear assembly are shown in Tables 1, 2, and 3.

TABLE 1. SINGLE WHEEL ASSEMBLY

Gross Weight		Tire Pressure	
lbs.	(kg)	psi	(MN/m^2)
30,000	(13 600)	75	(0.52)
45,000	(20 400)	90	(0.62)
60,000	(27 200)	105	(0.72)
75,000	(34 000)	120	(0.83)

TABLE 2. DUAL WHEEL ASSEMBLY

Gross Weight		Tire Pressure		Dual Spacing	
lbs.	(kg)	psi	(MN/m^2)	in.	(mm)
50,000	(22 700)	80	(0.55)	20	(51)
75,000	(34 000)	110	(0.76)	21	(53)
100,000	(45 400)	140	(0.97)	23	(58)
150,000	(68 000)	160	(1.10)	30	(76)
200,000	(90 700)	200	(1.38)	34	(86)

TABLE 3. DUAL TANDEM ASSEMBLY

Gross Weight		Tire Pressure		Dual Spacing		Tandem Spacing	
lbs.	(kg)	psi	(MN/m^2)	in.	(mm)	in.	(mm)
100,000	(45 400)	120	(0.83)	20	(51)	45	(114)
150,000	(68 000)	140	(0.97)	20	(51)	45	(114)
200,000	(90 700)	160	(1.10)	21	(53)	46	(117)
300,000	(136 100)	180	(1.24)	26	(66)	51	(130)
400,000	(181 400)	200	(1.38)	30	(76)	55	(140)

b. **Specific Design Curves.** Design curves for specific aircraft are presented in Chapter 3 for aircraft which have characteristics different from those assumed in Table 1, 2, or 3. Newer aircraft sometimes have different characteristics than those assumed for the generalized curves. The landing gear characteristics associated with each specific design curve is given on the curve.

2. **RIGID PAVEMENTS.** The design of rigid airport pavements is based on the Westergaard analysis of an edge loaded slab resting on a dense liquid foundation. The edge loading stresses are reduced by 25 percent to account for load transfer across joints. Two different cases of edge loading are covered by the design curves. Figures 3-17 through 3-29 assume the landing gear assembly is either tangent to a longitudinal joint or perpendicular to a transverse joint, whichever produces the largest stress. Figures 3-30 through 3-40 are for dual tandem assemblies and have been rotated through an angle to produce the maximum edge stress. Computer analyses were performed for angles from 0 to 90 degrees in 10-degree increments. Single and dual wheel assemblies were analyzed for loadings tangent to the edge only since the stress is maximum in that position. Sketches of the various assembly positions are shown in Figure 1.

a. **Load Repetitions.** Fatigue effects are considered in the design process by increasing slab thickness for increased numbers of load repetitions. Load repetitions are expressed in coverages. For rigid pavements, coverages equal the number of times a pavement slab experiences a maximum stress application due to applied traffic. One coverage occurs when each point in the pavement within the limits of the traffic lane has experienced a maximum stress, assuming the stress is equal under the full tire print. The coverage concept provides a means of normalizing pavement performance data which can consist of a variety of wheel sizes, spacings and loads for pavements of different cross sections. Each pass (departure) of an aircraft can be converted to coverages using a single pass-to-coverage ratio which is developed assuming a normal distribution and applying standard statistical techniques. The pass-to-coverage ratios used in developing the rigid pavement design curves in Chapter 3 are given in Table 4. Annual departures are converted to coverages assuming a 20-year design life. Coverages are determined by multiplying annual departures by 20 and dividing that product by the pass-to-coverage ratio shown in Table 4.

TABLE 4. PASS-TO-COVERAGE RATIOS FOR RIGID PAVEMENTS

Design Curve	Pass-to-Coverage Ratio
Single Wheel	5.18
Dual Wheel	3.48
Dual Tandem	3.68
A-300 Model B2	3.51
A-300 Model B4	3.45
B-747	3.70
B-757	3.88
B-767	3.90
C-130	4.15
DC 10-10	3.64
DC 10-30	3.38
L-1011	3.62

b. **Basic Thickness Calculation.** Pavement thickness requirements for 5,000 coverages are computed for various concrete strengths and subgrade moduli. Allowable concrete stress for 5,000 coverages is computed by dividing the concrete flexural strength by 1.3 (analogous to a safety factor). The thickness of pavement required to sustain 5,000 coverages of the design loading is considered to be 100 percent thickness.

c. **Thickness Adjustment.** The basic slab thicknesses are multiplied by the percent thickness shown in Figure 2 for other coverage levels. accordance with the fatigue curve developed by the Corps of Engineers from test track data and observation of in-service pavements. The fatigue relationship is applicable to the pavement structure; i.e., the slab and foundation are both included in the relationship. Any coverage level could have been selected as the 100 percent thickness level as long as the relative thicknesses for other coverage levels shown in Figure 2 is maintained.

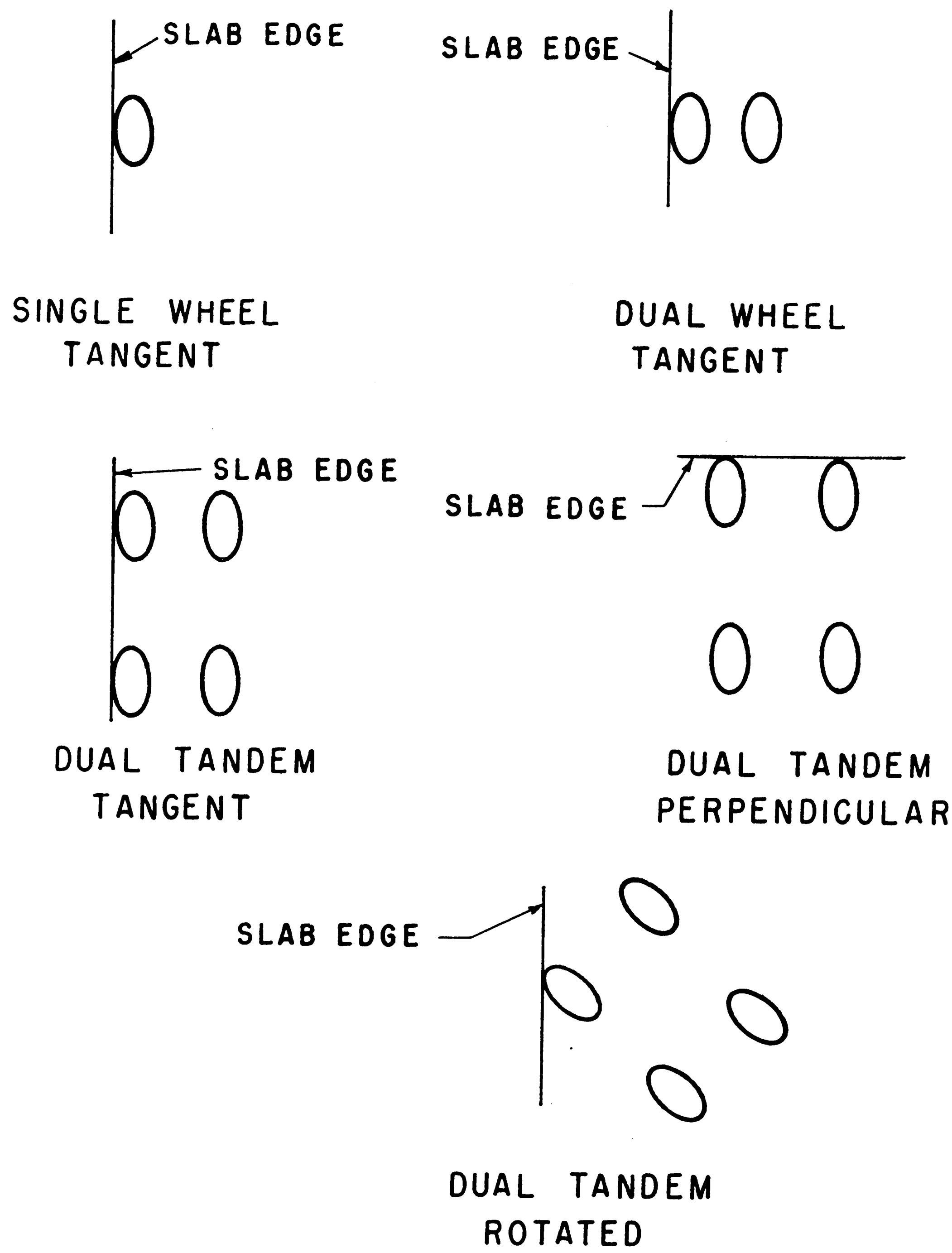


FIGURE 1. ASSEMBLY POSITIONS FOR RIGID PAVEMENT ANALYSIS

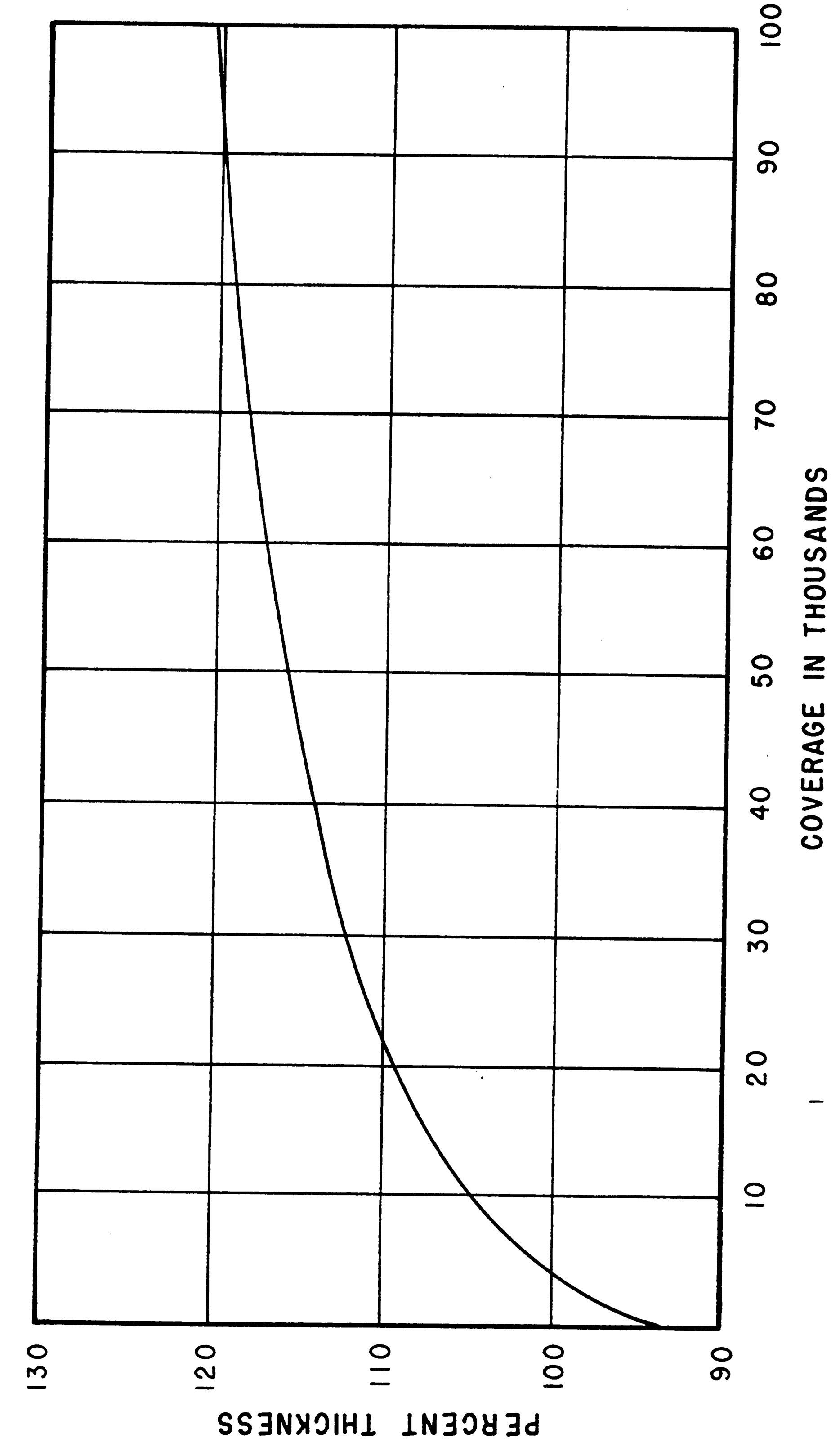


FIGURE 2. PERCENT THICKNESS VS. COVERAGES

3. FLEXIBLE PAVEMENTS. The design curves for flexible pavements in Chapter 3 of this circular are based on the CBR method of design. The CBR is the ratio of the load required to produce a specified penetration of a standard piston into the material in question to the load required to produce the same penetration in a standard well-graded, crushed limestone. Pavement thicknesses necessary to protect various CBR values from shear failure have been developed through test track studies and observations of in-service pavements. These thicknesses have been developed for single wheel loadings. Assemblies other than single wheel are designed by computing the equivalent single wheel load for the assembly based on deflection. Once the equivalent single wheel is established, the pavement section thickness can be determined from the relationships discussed above.

a. Load Repetitions. Load repetitions are indicated on the design curves in terms of annual departures. The annual departures are assumed to occur over a 20-year life. In the development of the design curves, departures are converted to coverages. For flexible pavements, coverage is a measure of the number of maximum stress applications that occur on the surface of the pavement due to the applied traffic. One coverage occurs when all points on the pavement surface within the traffic lane have been subjected to one application of maximum stress, assuming the stress is equal under the full tire print. Each pass (departure) of an aircraft can be converted to coverages using a single pass-to-coverage ratio which is developed assuming a normal distribution and applying standard statistical techniques. The pass-to-coverages ratios used in developing the flexible pavement design curves in Chapter 3 are given in Table 5. Annual departures are converted to coverages by multiplying by 20 and dividing that product by the pass-to-coverage ratios given in Table 5. Figure 3 shows the relationship between load repetition factor and coverages. The pavement section thickness determined in accordance with paragraph a. above is multiplied by the appropriate load repetition factor, Figure 3, to give the final pavement thickness required for various traffic levels.

**TABLE 5.
PASS-TO-COVERAGE
RATIOS FOR FLEXIBLE
PAVEMENTS**

Design Curve	Pass-to-Converge Ratio
Single Wheel	5.18
Dual Wheel	3.48
Dual Tandem	1.84
A-300 Model B2	1.76
A-300 Model B4	1.73
B-747	1.85
B-757	1.94
B-767	1.95
C-130	2.07
DC 10-10	1.82
DC 10-30	1.69
L-1011	1.81

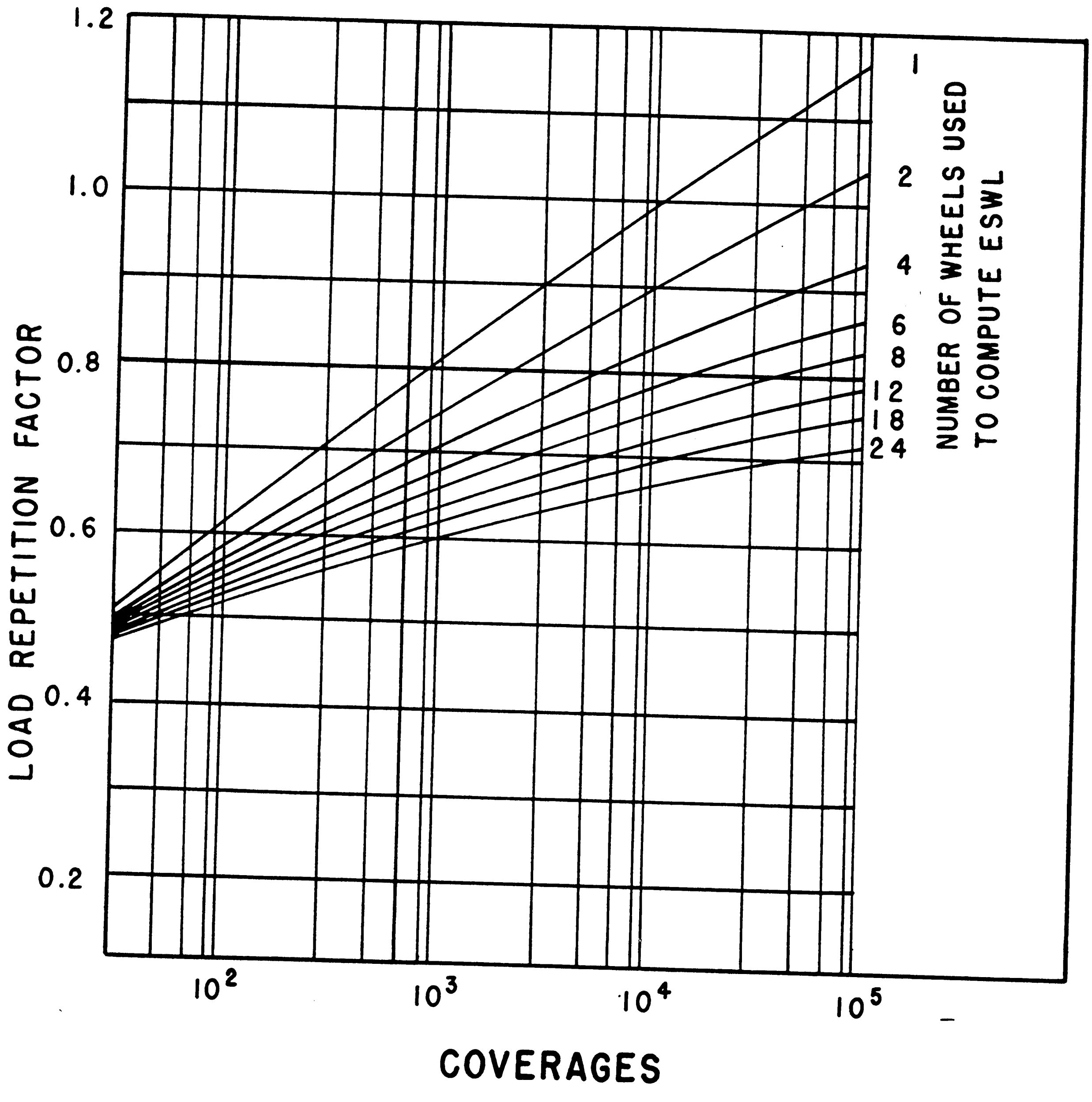


FIGURE 3. LOAD REPETITION FACTOR VS. COVERAGES

APPENDIX 3. DESIGN OF STRUCTURES FOR HEAVY AIRCRAFT

1. **BACKGROUND.** Airport structures such as culverts and bridges are usually designed to last for the foreseeable future of the airport. Information concerning the landing gear arrangement of future heavy aircraft is speculative. It may be assumed with sufficient confidence that strengthening of pavements to accommodate future aircraft can be performed without undue problems. Strengthening of structures, however, may prove to be extremely difficult, costly, and time-consuming. Point loadings on some structures may be increased; while on overpasses, the entire aircraft weight may be imposed on a deck span, pier, or footing.

2. **RECOMMENDED DESIGN PARAMETERS.**

a. **Structural Considerations.** For many structures the design is highly dependent upon the aircraft gear configuration. Our assessment indicates that three basic configurations, shown in Figure 1, will, if all are considered in the design of the bridge components, provide sufficient support for any aircraft which may be forthcoming. These consist of two areas enclosing eight wheels each, or 16 wheels per aircraft comprising the main gear. Nose gears, as such, are not considered, except as they occur in the static load. The "area" dimensions are 6 to 8 feet by 20 feet (2-3 m by 6 m) each supporting half of the aircraft gross weight. Wheel prints are uniformly spaced within their respective areas.

b. **Foundation Design.** Foundation design will vary with soil type and depth. No departure from accepted methodology is anticipated; except that for shallow structures, such as inlets and culverts, the concentrated loads may require heavier and wider spread footings than those presently provided by the structural standards in current use. For buried structures, such as culverts, the following guidance from AASHTO is recommended.

(1) When the depth of fill is less than 2 feet the wheel loads shall be treated as concentrated loads.

(2) When the depth of fill is 2 feet or more, wheel loads shall be considered as uniformly distributed over a square with sides equal to 1-3/4 times the depth of the fill. When such areas from several concentrations overlap, the total load shall be uniformly distributed over the area defined by the outside limits of the individual areas, but the total width of distribution shall not exceed the total width of the supporting slab.

c. **Loads.** It should be noted that all loads discussed herein are to be considered as dead load plus live loads. The design of structures subject to direct wheel loads should also anticipate braking loads as high as 0.7 G (for no-slip brakes).

d. **Direct Loading.** Decks and covers subject to direct heavy aircraft loadings such as manhole covers, inlet grates, utility tunnel roofs, bridges, etc., should be designed for the following loadings:

(1) Manhole covers for 100,000 lb. (45 000 kg) wheel loads with 250 psi (1.72 MPa) tire pressure.

(2) For spans of 2 feet (0.6 m) or less in the least direction, a uniform live load of 250 psi (1.72 Mpa).

(3) For spans of 2 feet (0.6 m) or greater in the least direction, the design shall be based on the number of wheels which will fit the span. Wheel loads of 50,000 to 75,000 pounds (22 700 to 34 000 kg) should be considered.

(4) Special consideration shall be given to structures that will be required to support both in-line and diagonal traffic lanes, such as diagonal taxiways or apron taxi routes. If structures require expansion joints, load transfer may not be possible.

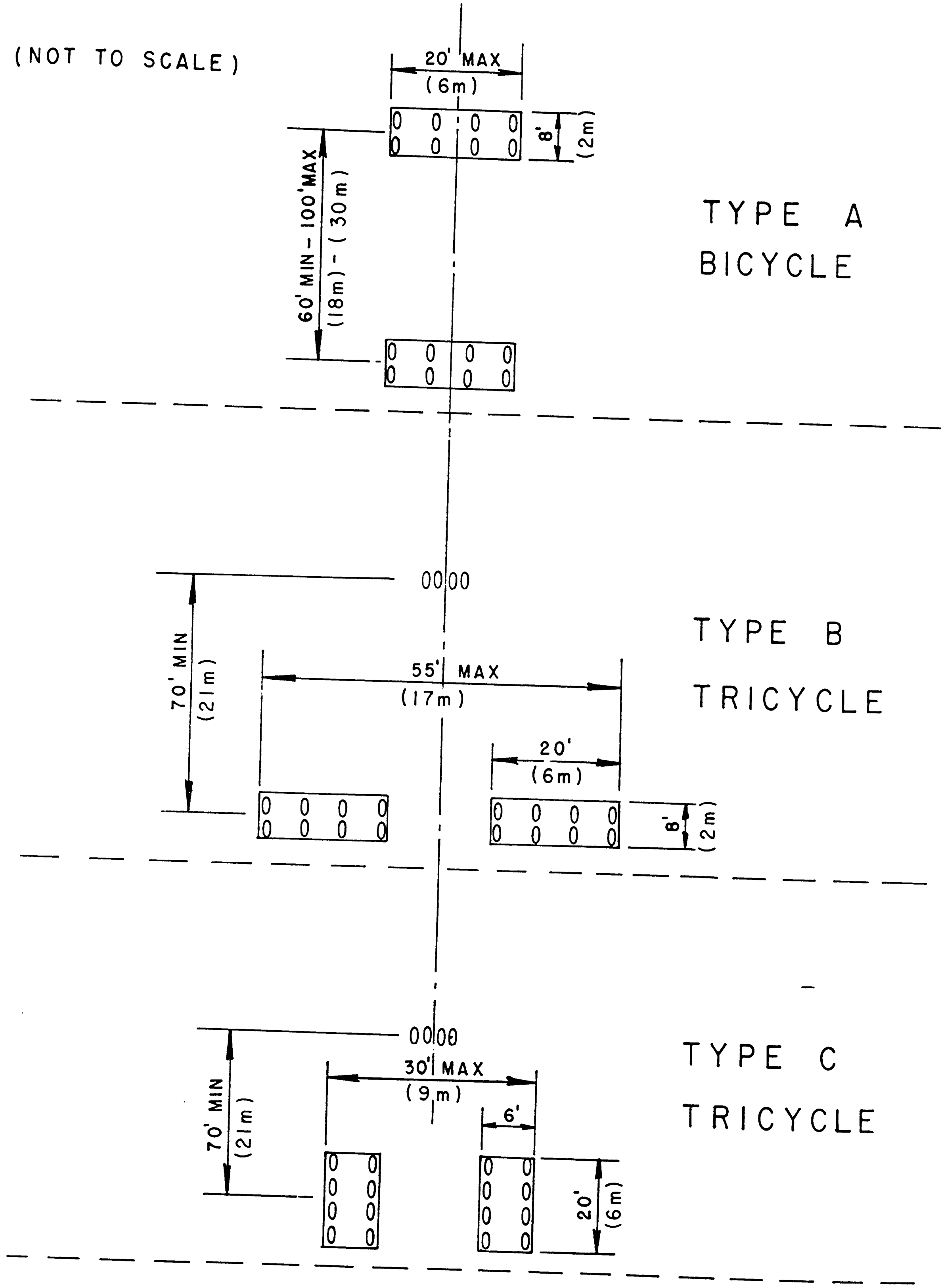


FIGURE 1. TYPICAL GEAR CONFIGURATIONS FOR DESIGN OF STRUCTURES

APPENDIX 4. RELATED READING MATERIAL

1. The latest issuance of the following free publications may be obtained from the Department of Transportation, Utilization and Storage Section, M-443.2, Washington, D.C. 20590. Advisory Circular 00-2, updated triennially, contains the listing of all current issuances of these circulars and changes thereto.

a. AC 00-2, Federal Register, Advisory Circular Checklist and Status of Federal Aviation Regulations.

b. AC 150/5320-12, Measurement, Construction, and Maintenance of Skid Resistant Airport Pavement Surfaces.

c. AC 150/5335-5, Standardized Method of Reporting Airport Pavement Strength.

d. AC 150/5370-11, Use of Nondestructive Testing Devices in the Evaluation of Airport Pavements.

e. AC 150/5380-6, Guidelines and Procedures for Maintenance of Airport Pavements.

f. AC 1520/5300-9, Predesign, Prebid, and Preconstruction Conferences for Airport Grant Projects.

2. The following advisory circulars which can be found in AC 00-2 may be obtained from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402. Use the Superintendent of Documents stock number when ordering, along with the AC number and title. Send a check or money order in the amount listed for each document. No. c.o.d. orders are accepted.

a. AC 150/5300-13, Airport Design.

b. AC 150/5320-5, Airport Drainage.

c. AC 150/5370-10, Standards for Specifying Construction of Airports.

3. Copies of the following reports may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

a. FAA-RD-73-169, Review of Soil Classification Systems Applicable to Airport Pavement Design, May 1974, by Yoder; AD-783-190.

b. FAA-RD-74-30, Design of Civil Airfield Pavement for Seasonal Frost and Permafrost Conditions, October 1974, by Berg; ADA-006-284.

c. FAA-RD-74-36, Field Survey and Analysis of Aircraft Distribution on Airport Pavements, February 1975, by Ho Sang; ADA-011-488.

d. FAA-RD-76-66, Design and Construction of Airport Pavements on Expansive Soils, January 1976, by McKeen; ADA-28-094.

e. FAA-RD-73-198-I, Design and Construction and Behavior Under Traffic of Pavement Test Sections, June 1974, by Burns, Rone, Brabston, Ulery; AD-785-024.

f. FAA-RD-74-33, III, Design Manual for Continuously Reinforced Concrete Pavements, May 1974, by Treybig, McCullough, Hudson; AD-780-512.

g. FAA-RD-75-110-II, Methodology for Determining, Isolating and Correcting Runway Roughness, June 1977, by Seeman, Nielsen; ADA-44-378.

h. FAA-RD-73-198-III, Design and Construction of MESL, December 1974 by Hammitt; AD-005-893.

i. FAA-RD-76-179, Structural Design of Pavements for Light Aircraft December 1976, by Ladd, Parker, Percira; ADA-041-300.

j. FAA-RD-74-39, Pavement Response to Aircraft Dynamic Loads, Volume II - Presentation and Analysis of Data, by Ledbetter; ADA-22-806.

k. FAA-RD-81-78, Economic Analysis of Airport Pavement Rehabilitation Alternatives, October 1981, by Epps and Wootan.

4. Copies of ASTM standards may be obtained from the American Society for Testing and Materials, 1916 Race Street, Philadelphia, Pennsylvania 19103.

5. Copies of AASHTO standards may be obtained from the American Association of State Highway and Transportation Officials, 314 National Press Building, Washington, D.C. 20004.

6. Copies of the following publications may be obtained from: Commander, U.S. Army, A.G. Publication Center, 1655 Woodson Road, St. Louis, Missouri 63114.

a. TM5-824-2, Flexible Airfield Pavements, Department of the Army Technical Manual, February 1969.

b. TM5-824-3, Rigid Pavements for Airfields other than Army, Departments of the Army and the Air Force, Technical Manual, December 1970.

c. TM5-818-2, Pavement Design for Frost Conditions, Department of the Army, Technical Manual, July 1965.

7. Copies of MS-11, Full Depth Asphalt Pavements for Air Carrier Airports, January 1973, IS-154, Full Depth Asphalt Pavements for General Aviation, January 1973, and MS-10, Soils Manual, Fourth Edition (1986), may be obtained from the Asphalt Institute, Asphalt Institute Building, College Park, Maryland 20740.

8. Copies of Engineering Bulletin, Design of Concrete Airport Pavement, by Robert G. Packard can be obtained from the Portland Cement Association, Old Orchard Road, Skokie, Illinois 60076.

9. Copies of Special Technical Publication M-5, The Estimation of Concrete Flexural strength from Other Types of Strength Tests, by W. Charles Greer can be obtained from Director of Publications, Law Engineering Testing Company, Corporate Office, 1140 Hammond Drive, Atlanta, GA 30328.

10. Copies of FHWA-HI-90-001, Geotextile Design and Construction Guidelines, October 1989, obtained from the Department of Transportation, Federal Highway Administration, Turner Fairbank Highway Research Center, 6300 Georgetown Pike, McLean, Virginia, 22101-229.

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